Chapter 15 Contents

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Chapter 15 Abutments, Retaining Walls, and Reinforced Slopes

15.1 Introduction

Abutments for bridges have components of both foundation design and wall design. This chapter addresses the earth pressures acting on the abutments as well as retaining walls and reinforced slopes. Retaining walls and reinforced slopes are typically included in projects to minimize construction in wetlands, to widen existing facilities, and to minimize the amount of right of way needed in urban environments. Projects modifying existing facilities often need to modify or replace existing retaining walls or widen abutments for bridges. All abutments, walls, and reinforced slopes within WSDOT right of way shall be designed and constructed in accordance with AASHTO requirements and this manual.

Retaining walls and reinforced slopes have many benefits associated with their use. Unfortunately, there also tends to be confusion regarding when they should be incorporated into a project, what types are appropriate, how they are designed, who designs them, and how they are constructed. The rolls and responsibilities of the various WSDOT offices and those of the Department's consultants further confuse the issue of retaining walls and reinforced slopes, as many of the rolls and responsibilities overlap or change depending on the wall type. All abutments, retaining walls, and reinforced slopes within WSDOT Right of Way or whose construction is administered by WSDOT shall be designed in accordance with the WSDOT Geotechnical Design Manual (GDM) and the following documents:

- WSDOT LRFD Bridge Design Manual
- WSDOT Design Manual M 22-01
- WSDOT Standard Plans for Road, Bridge, and Municipal Construction M 21-01
- AASHTO LRFD Bridge Design Specifications, U.S.

The most current versions of the above referenced manuals including all interims or design memoranda modifying the manuals shall be used. In the case of conflict or discrepancy between manuals, the following hierarchy shall be used: Those manuals listed first shall supercede those listed below in the list.

The following manuals provide additional design and construction guidance for retaining walls and reinforced slopes and should be considered supplementary to the **WSDOT GDM** and the manuals and design specifications listed above:

- Lazarte, C. A., Elias, V., Espinoza, R. D., Sabatini, P. J., 2003. Geotechnical Engineering Circular No. 7, Soil Nail Walls, U.S. Department of Transportation, Federal Highway Administration, FHWA-IF-03-017, 305 pp.
- Porterfield, J. A., Cotton, D. A., Byrne, R. J., 1994, Soil Nail Walls-Demonstration Project 103, Soil Nailing Field Inspectors Manual, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-93-068, 86 pp.
- Cheney, R., and Chassie, R. 2000. Soils and Foundations Workshop Reference Manual. Washington, DC, National Highway Institute Publication NHI-00-045, Federal Highway Administration.
- Elias, V., and Christopher, B.R., and Berg, R. R., 2001, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Design and Construction Guidelines, No. FHWA-NHI-00-043, Federal Highway Administration, 394 pp..
- Sabatini, P. J., Pass, D. G., and Bachus, R. C., 1999, Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems, FHWA-IF-99-015, 281 pp.

15.2 Definitions

The various walls and wall systems can be categorized based on how they are incorporated into construction contracts. Standard Walls comprise the first category and are the easiest to implement. Standard walls are those walls for which standard designs are provided in the WSDOT Standard Plans. The internal stability design and the external stability design for overturning and sliding stability have alrearesistancecapacity, and settlement must be determined for each standard-design wall location. All other walls are nonstandard, as they are not included in the Standard Plans.

Nonstandard walls may be further subdivided into proprietary or nonproprietary. Nonstandard, proprietary walls are patented or trademarked wall systems designed and marketed by a wall manufacturer. The wall manufacturer is responsible for internal and external stability, except bearing resistance, settlement, and overall slope stability, which are determined by the geotechnical designer. Nonstandard, nonproprietary walls are not patented or trade marked wall systems. However, they may contain proprietary elements. An example of this would be a gabion basket wall. The gabion baskets themselves are a proprietary item. However, the gabion manufacturer provides gabions to a consumer, but does not provide a designed wall. It is up to the consumer to design the wall and determine the stable stacking arrangement of the gabion baskets. Nonstandard, nonproprietary walls are fully designed by the geotechnical designer and, if structural design is required, by the structural designer. Reinforced slopes are similar to nonstandard, nonproprietary walls in that the geotechnical designer is responsible for the design, but the reinforcing may be a proprietary item.

A number of proprietary wall systems have been extensively reviewed by the Bridge and Structures Office and the HQ Geotechnical Division. This review has resulted in WSDOT preapproving some proprietary wall systems. The design procedures and wall details for these preapproved wall systems have been agreed upon between WSDOT and the proprietary wall manufacturers. This allows the manufacturers to competitively bid a particular project without having a detailed wall design provided in the contract plans. Note that proprietary wall manufacturers may produce several retaining wall options, and not all options from a given manufacturer have been preapproved. The Bridge and Structures Office shall be contacted to obtain the current listing of preapproved proprietary systems prior to including such systems in WSDOT projects. A listing of the preapproved wall systems, as of the current publication date for this manual, is provided in **WSDOT GDM Appendix 15-D**. Specific preapproved details and system specific design requirements for each wall system are also included as appendices to **WSDOT GDM Chapter 15**. Incorporation of nonpreapproved systems requires the wall supplier to completely design the wall prior to advertisement for construction. All of the manufacturer's plans and details would need to be incorporated into the contract documents. Several manufacturers may need to be contacted to maintain competitive bidding. More information is available in Chapters 510 and 1130 of the WSDOT Design Manual M 22-01.

If it is desired to use a non-preapproved proprietary retaining wall or reinforced slope system, review and approval for use of the wall or slope system on WSDOT projects shall be based on the "HITEC Submittal Protocol Checklist" for walls, as modified for WSDOT use, provided in **WSDOT GDM Appendix 15-C**. The wall or reinforced slope system, and its design and construction, shall meet the requirements provided in this manual, including **WSDOT GDM Appendix 15-A**. For MSE walls, the wall supplier shall demonstrate in the wall submittal that the proposed wall system can meet the facing performance tolerances provided in **WSDOT GDM Appendix 15-A** through calculation, construction technique, and actual measured full scale performance of the wall system proposed.

15.3 Required Information

15.3.1 Site Data and Permits

The WSDOT State Design Manual discusses site data and permits required for design and construction. In addition, Chapters 510 and 1130 provide specific information relating to geotechnical work and retaining walls.

15.3.2 Geotechnical Data Needed for Retaining Wall and Reinforced Slope Design

The project requirements, site, and subsurface conditions should be analyzed to determine the type and quantity of information to be developed during the geotechnical investigation. It is necessary to:

- Identify areas of concern, risk, or potential variability in subsurface conditions
- Develop likely sequence and phases of construction as they may affect retaining wall and reinforced slope selection
- Identify design and constructability requirements or issues such as:
  - *Surcharge loads from adjacent structures*
  - *Backslope and toe slope geometries*
  - *Right of way restrictions*
  - *Materials sources*
  - *Easements*
  - *Excavation limits*
  - *Wetlands*
  - *Construction Staging*
- Identify performance criteria such as:
  - *Tolerable settlements for the retaining walls and reinforced slopes*
  - *Tolerable settlements of structures or property being retained*
  - *Impact of construction on adjacent structures or property*
  - *Long-term maintenance needs and access*
- Identify engineering analyses to be performed:
  - *Bearing resistance*
  - *Settlement*
  - *Global stability*
  - *Internal stability*
- Identify engineering properties and parameters required for these analyses
- Identify the number of tests/samples needed to estimate engineering properties

Table 15.1 provides a summary of information needs and testing considerations for retaining walls and reinforced slope design.

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Fill Walls/ Reinforced Soil Slopes	• internal stability • external stability • limitations on rate of construction • settlement • horizontal deformation? • lateral earth pressures? • bearing capacity? • chemical compatibility with soil, groundwater, and wall materials? • pore pressures behind wall • borrow source evaluation (available quantity and quality of borrow soil) • liquefaction • potential for subsidence (karst, mining, etc.) • constructability • scour	• subsurface profile (soil, ground water, rock) • horizontal earth pressure coefficients • interface shear strengths • foundation soil/ wall fill shear strengths? • compressibility parameters? (including consolidation, shrink/swell potential, and elastic modulus) • chemical composition of fill/ foundation soils? • hydraulic conductivity of soils directly behind wall? • time-rate consolidation parameters? • geologic mapping including orientation and characteristics of rock discontinuities? • design flood elevations • seismicity	• SPT • CPT • dilatometer • vane shear • piezometers • test fill? • nuclear density? • pullout test (MSEW/RSS) • rock coring (RQD) • geophysical testing	• 1-D Oedometer • triaxial tests • unconfined compression • direct shear tests • grain size distribution • Atterberg Limits • specific gravity • pH, resistivity, chloride, and sulfate tests? • moisture content? • organic content • moisture-density relationships • hydraulic conductivity

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Cut Walls	• internal stability • external stability • excavation stability • dewatering • chemical compatibility of wall/soil • lateral earth pressure • down-drag on wall • pore pressures behind wall • obstructions in retained soil • liquefaction • seepage • potential for subsidence (karst, mining, etc.) • constructability	• subsurface profile (soil, ground water, rock) • shear strength of soil • horizontal earth pressure coefficients • interface shear strength (soil and reinforcement) • hydraulic conductivity of soil • geologic mapping including orientation and characteristics of rock discontinuities • seismicity	• test cut to evaluate stand-up time • well pumping tests • piezometers • SPT • CPT • vane shear • dilatometer • pullout tests (anchors, nails) • geophysical testing	• triaxial tests • unconfined compression • direct shear • grain size distribution • Atterberg Limits • specific gravity • pH, resistivity tests • organic content • hydraulic conductivity • moisture content • unit weight

Table 15.1 Summary of information needs and testing considerations.

WSDOT GDM Chapter 5 covers requirements for how the results from the field investigation, the field testing, and laboratory testing are to be used to establish properties for design. The specific tests and field investigation requirements needed for foundation design are described in the following sections.

15.3.3 Site Reconnaissance

For each abutment, retaining wall, and reinforced slope, the geotechnical designer should perform a site review and field reconnaissance. The geotechnical designer should be looking for specific site conditions that could influence design, construction, and performance of the retaining walls and reinforced slopes on the project. This type of review is best performed once survey data has been collected for the site and digital terrain models, cross-sections, and preliminary wall profiles have been generated by the civil engineer (e.g., region project engineer). In addition, the geotechnical designer should have access to detailed plan views showing existing site features, utilities, proposed construction, and right or way limits. With this information, the geotechnical designer can review the wall/slope locations making sure that survey information agrees reasonably well with observed site topography. The geotechnical designer should observe where utilities are located, as they will influence where field exploration can occur and they may affect design or constructability. The geotechnical designer should look for indications of soft soils or unstable ground. Items such as hummocky topography, seeps or springs, pistol butted trees, and scarps, either old or new, need to be investigated further. Vegetative indicators such as equisetum (horsetails), cat tails, black berry, or alder can be used to identify soils that are wet or unstable. A lack of vegetation can also be an indicator of recent slope movement. In addition to performing a basic assessment of site conditions, the geotechnical designer should also be looking for existing features that could influence design and construction such as nearby structures, surcharge loads, and steep back or toe slopes. This early in design, it is easy to overlook items such as construction access, materials sources, and limits of excavation. The geotechnical designer needs to be cognizant of these issues and should be identifying access and excavation issues early, as they can affect permits and may dictate what wall type may or may not be used.

15.3.4 Field Exploration Requirements

A soil investigation and geotechnical reconnaissance is critical for the design of all abutments, retaining walls, or reinforced slopes. The stability of the underlying soils, their potential to settle under the imposed loads, the usability of any existing excavated soils for wall/reinforced slope backfill, and the location of the ground water table are determined through the geotechnical investigation. All abutments, retaining, walls and reinforced slopes regardless of their height require an investigation of the underlying soil/rock that supports the structure. Abutments shall be investigated like other bridge piers in accordance with **WSDOT GDM Chapter 8**.

Retaining walls and reinforced slopes that are equal to or less than 10 feet in exposed height as measured vertically from wall bottom to top or from slope toe to crest, as shown in **Figure 15.1**, shall be investigated in accordance with this manual. For all retaining walls and reinforced slopes greater than 10 feet in exposed height, the field exploration shall be completed in accordance with the AASHTO LRFD Bridge Design Specifications and this manual.

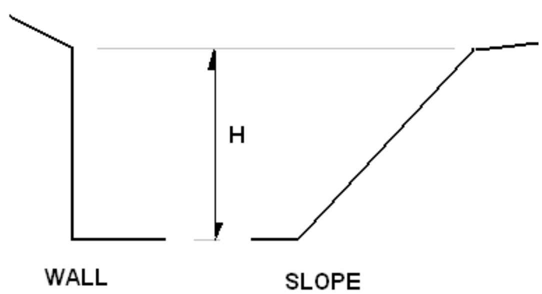

Figure 15.1 Exposed height (H) for a retaining wall or slope.

Explorations consisting of geotechnical borings, test pits, hand holes, or a combination thereof shall be performed at each wall or slope location. Geophysical testing may be used to supplement the subsurface exploration and reduce the requirements for borings. If the geophysical testing is done as a first phase in the exploration program, it can also be used to help develop the detailed plan for second phase exploration. As a minimum, the subsurface exploration and testing program should obtain information to analyze foundation stability and settlement with respect to:

- Geological formation(s)
- Location and thickness of soil and rock units
- Engineering properties of soil and rock units, such as unit weight, shear strength and compressibility
- Ground water conditions

- Ground surface topography
- Local considerations, (e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential)

In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to perform more investigation to capture variations in soil and/or rock type and to assess consistency across the site area. In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will affect the soil and/or rock mass in order to optimize the exploration. The following minimum guidelines for frequency and depth of exploration shall be used. Additional exploration may be required depending on the variability in site conditions, wall/slope geometry, wall/slope type, and the consequences should a failure occur.

15.3.4.1 Exploration Type, Depth, and Spacing

Generally, walls 10 feet or less in height, constructed over average to good soil conditions (e.g., non-liquefiable, medium dense to very dense sand, silt or gravel, with no signs of previous instability) will require only a basic level of site investigation. A geologic site reconnaissance (see **WSDOT GDM Chapter 2**), combined with widely spaced test pits, hand holes, or a few shallow borings to verify field observations and the anticipated site geology may be sufficient, especially if the geology of the area is well known, or if there is some prior experience in the area.

The geotechnical designer should investigate to a depth below bottom of wall or reinforced slope at least to a depth where stress increase due to estimated foundation load is less than 10% of the existing effective overburden stress and between 1 and 2 times the exposed height of the wall or slope. Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g. peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock). Hand holes and test pits should be used only where medium dense to dense granular soil conditions are expected to be encountered within limits that can be reasonably explored using these methods, approximately 10 feet for hand holes and 15 feet for test pits, and that based on the site geology there is little risk of an unstable soft or weak layer being present that could affect wall stability.

For retaining walls and reinforced slopes less than 100 feet in length, the exploration should occur approximately midpoint along the alignment or where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e. where the height is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues.

For retaining walls and slopes more than 100 feet in length, exploration points should be spaced no more than 500 feet in uniform, dense soil conditions and should be spaced at 100 to 200 ft in typical soil conditions. Even closer spacing should be used in highly variable and potentially unstable soil conditions. Where possible, locate at least one boring where the maximum height occurs. Explorations should be completed on the alignment of the wall face or approximately midpoint along the reinforced slope, i.e. where the height is 0.5H. Additional borings to investigate the toe slope for walls or the toe catch for reinforced slopes may be required to assess overall stability issues.

A key to the establishment of exploration frequency for walls is the potential for the subsurface conditions to impact the construction of the wall, the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project.

15.3.4.2 Walls and Slopes Requiring Additional Exploration

15.3.4.2.1 Soil Nail Walls

Soil nail walls should have additional geotechnical borings completed to explore the soil conditions within the soil nail zone. The additional exploration points shall be at a distance of 1.0 to 1.5 times the height of the wall behind the wall to investigate the soils in the nail zone. Borings should be spaced no more than 500 feet in uniform, dense soil conditions and should be spaced at 100 to 200 ft in typical soil conditions. Even closer spacing should be used in highly variable and potentially unstable soil conditions. The depth of the borings shall be sufficient to explore the full depth of soils where nails are likely to be installed, and deep enough to address overall stability issues.

In addition, each soil nail wall should have at least one test pit excavated to evaluate stand-up time of the excavation face. The test pit shall be completed outside the nail pattern, but as close as practical to the wall face to investigate the stand-up time of the soils that will be exposed at the wall face during construction. The test pit shall remain open at least 24 hours and shall be monitored for sloughing, caving, and groundwater seepage. A test pit log shall be prepared and photographs should be taken immediately after excavation and at 24 hours. If variable soil conditions are present along the wall face, a test pit in each soil type should be completed. The depth of the test pits should be at least twice the vertical nail spacing and the length along the trench bottom should be at least one and a half times the excavation depth to minimize soil-arching effects. For example, a wall with a vertical nail spacing of 4 feet would have a test pit 8 feet deep and at least 12 feet in length at the bottom of the pit.

15.3.4.2.2 Walls with Ground Anchors or Deadmen Anchors

Walls with ground anchors or deadman anchors should have additional geotechnical borings completed to explore the soil conditions within the anchor/deadman zone. These additional borings should be spaced no more than 500 feet in uniform, dense soil conditions and should be spaced at 100 to 200 ft in typical soil conditions. Even closer spacing should be used in highly variable and potentially unstable soil conditions. The borings should be completed outside the no-load zone of the wall in the bond zone of the anchors or at the deadman locations. The depth of the borings shall be sufficient to explore the full depth of soils where anchors or deadmen are likely to be installed, and deep enough to address overall stability issues.

15.3.4.2.3 Wall or Slopes with Steep Back Slopes or Steep Toe Slopes

Walls or slopes that have a back slopes or toe slopes that exceed 10 feet in slope length and that are steeper than 2H:1V should have at least one hand hole, test pit, or geotechnical boring in the backslope or toe slope to define stratigraphy for overall stability analysis and evaluate bearing resistance. The exploration should be deep enough to address overall stability issues. Hand holes and test pits should be used only where medium dense to dense granular soil conditions are expected to be encountered within limits that can be reasonably explored using these methods, approximately 10 feet for hand holes and 20 feet for test pits.

15.3.5 Field, Laboratory, and Geophysical Testing for Abutments, Retaining Walls, and Reinforced Slopes

The purpose of field and laboratory testing is to provide the basic data with which to classify soils and to estimate their engineering properties for design. Often for abutments, retaining walls, and reinforced slopes, the backfill material sources are not known or identified during the design process. For example, mechanically stabilized earth walls are commonly constructed of backfill material that is provided by the Contractor during construction. During design, the material source is not known and hence materials cannot be tested. In this case, it is necessary to design using commonly accepted values for regionally available materials and ensure that the contract will require the use of materials meeting or exceeding these assumed properties.

For abutments, the collection of soil samples and field testing shall be in accordance with **WSDOT GDM Chapters 2, 5, and 8.**

For retaining walls and reinforced slopes, the collection of soil samples and field testing are closely related. **WSDOT GDM Chapter 5** provides the minimum requirements for frequency of field tests that are to be performed in an exploration point. As a minimum, the following field tests shall be performed and soil samples shall be collected:

In geotechnical borings, soil samples shall be taken during the Standard Penetration Test (SPT). Fine grained soils or peat shall be sampled with 3-inch Shelby tubes or WSDOT Undisturbed Samplers if the soils are too stiff to push 3-inch Shelby tubes. All samples in geotechnical borings shall be in accordance with **WSDOT GDM Chapters 2 and 3**.

In hand holes, sack soil samples shall be taken of each soil type encountered, and WSDOT Portable Penetrometer tests shall be taken in lieu of SPT tests. The maximum vertical spacing between portable penetrometer tests should be 5 feet.

In test pits, sack soil samples shall be taken from the bucket of the excavator, or from the spoil pile for each soil type encountered once the soil is removed from the pit. WSDOT Portable Penetrometer tests may be taken in the test pit. However, no person shall enter a test pit to sample or perform portable penetrometer tests unless there is a protective system in place in accordance with WAC 296-155-657.

In soft soils, CPT tests or insitu vane shear tests may be completed to investigate soil stratigraphy, shear strength, and drainage characteristics.

All soil samples obtained shall be reviewed by a geotechnical engineer or engineering geologist. The geotechnical designer shall group the samples into stratigraphic units based on consistency, color, moisture content, engineering properties, and depositional environment. At least one sample from each stratigraphic unit should be tested in the laboratory for Grain Size Distribution, Moisture Content, and Atterberg Limits (fine grained soils only). Additional tests, such as Loss on Ignition, pH, Resistivity, Sand Equivalent, or Hydrometer may be performed.

Walls that will be constructed on compressible or fine grained soils should have undisturbed soil samples available for laboratory testing, e.g. shelby tubes or WSDOT undisturbed samples. Consolidation tests and Unconsolidated Undrained (UU) triaxial tests should be performed on fine grained or compressible soil units. Additional tests such as Consolidated Undrained (CU), Direct Shear, or Lab Vane Shear may be performed to estimate shear strength parameters and compressibility characteristics of the soils.

Geophysical testing may be used for establishing stratification of the subsurface materials, the profile of the top of bedrock, depth to groundwater, limits of types of soil deposits, the presence of voids, anomalous deposits, buried pipes, and depths of existing foundations. Data from Geophysical testing shall always be correlated with information from direct methods of exploration, such as SPT, CPT, etc.

15.3.6 Groundwater

One of the principal goals of a good field reconnaissance and field exploration is to accurately characterize the groundwater in the project area. Groundwater affects the design, performance, and constructability of project elements. Installation of piezometer(s) and monitoring is usually necessary to define groundwater elevations. Groundwater measurements shall be conducted in accordance with **WSDOT GDM Chapter 2**, and shall be assessed for each wall. In general, this will require at least one groundwater measurement point for each wall. If groundwater has the potential to affect wall performance or to require special measures to address drainage to be implemented, more than one measurement point per wall will be required.

15.4 General Design Requirements

15.4.1 Design Methods

The AASHTO LRFD Bridge Design Specifications shall be used for all abutments and retaining walls addressed therein. The walls shall be designed to address all applicable limit states (strength, service, and extreme event). Rock walls, reinforced slopes, and soil nail walls are not specifically addressed in the AASHTO specifications, and shall be designed in accordance with this manual. Many of the FHWA manuals used as WSDOT design references were not developed for LRFD design. For those wall types (and including reinforced slopes) for which LRFD procedures are not available, allowable stress design procedures included in this manual, either in full or by reference, shall be used, again addressing all applicable limit states.

The load and resistance factors provided in the AASHTO LRFD Specifications have been developed in consideration of the inherent uncertainty and bias of the specified design methods and material properties, and the level of safety used to successfully construct thousands of walls over many years. These load and resistance factors shall only be applied to the design methods and material resistance estimation methods for which they are intended, if an option is provided in this manual or the AASHTO LRFD specifications to use methods other than those specified herein or in the AASHTO LRFD specifications. For estimation of soil reinforcement pullout, the resistance factors provided are to be used only for the default pullout methods provided in the AASHTO LRFD specifications. If wall system specific pullout resistance estimation methods are used, resistance factors shall be developed statistically using reliability theory to produce a probability of failure P_f of approximately 1 in 100 or smaller. Note that in some cases, Section 11 of the AASHTO LRFD Bridge Design Specifications refers to AASHTO LRFD Section 10 for wall foundation design and the resistance factors for foundation design. In such cases, the design methodology and resistance factors provided in the **WSDOT GDM Chapter 8** shall be used instead of the resistance factors in AASHTO LRFD Section 10.

It is recognized that many of the proprietary wall suppliers have not fully implemented the LRFD approach for the design of their wall system(s). The approved details for the currently preapproved proprietary wall systems have been developed in accordance with the AASHTO Standard Specifications for Highway Bridges (**2002**). WSDOT will allow a grace period for the wall systems preapproved on or before December 1, 2004, and have remained in approved status until the present, regarding the implementation of the LRFD approach. In those cases, the AASHTO Standard Specifications for Highway Bridges (**2002**), as modified in the **WSDOT GDM**, may be used for the design of those systems until such time that WSDOT decides to end the grace period.

For walls with a traffic barrier, design of the traffic barrier and the distribution of the applied impact load to the wall top shall be as described in the AASHTO Standard Specifications for Highway Bridges **(2002)**, Article 5.8.12.2, for both AASHTO Standard Specification wall designs and AASHTO LRFD Specification designs.

15.4.2 Special Requirements

All walls shall meet the requirements in the State Design Manual for layout and geometry. All walls shall be designed and constructed in accordance with the Standard Specifications, General Special Provisions, and Standard Plans. Specific design requirements for tiered walls, back-to-back walls, and MSE wall supported abutments are provided in the AASHTO LRFD Bridge Design Specifications (for preapproved proprietary wall systems, alternatively in the **AASHTO Standard Specifications for Highway Bridges, 2002**), and by reference in those design specifications **Elias, et al. (2001)**.

15.4.2.1 Tiered Walls

Walls that retain other walls or have walls as surcharges require special design to account for the surcharge loads from the upper wall. Proprietary wall systems may be used for the lower wall, but proprietary walls shall not be considered preapproved in this case. Chapter 1130 of the WSDOT Design Manual discusses the requirements for utilizing non-preapproved proprietary walls on WSDOT projects. If the upper wall is proprietary, a preapproved system may be used provided it meets the requirements for preapproval and does not contain significant structures or surcharges within the wall reinforcing.

15.4.2.2 Back-to-Back Walls

The face-to-face dimension for back-to-back sheetpile walls used as bulkheads for waterfront structures must exceed the maximum exposed height of the walls. Bulkhead walls may be cross braced or tied together provided the tie rods and connections are designed to carry twice the applied loads.

The face to face dimension for back to back Mechanically Stabilized Earth (MSE) walls shall be 1.1 times the average height of the MSE walls or greater. Back-to-back MSE walls with a width/height ratio of less than 1.1 shall not be used unless approved by the State Geotechnical Engineer and the Bridge Design Engineer. The maximum height for back-to-back MSE wall installations is 30 feet. The soil reinforcement for back-to-back MSE walls may be connected to both faces, i.e., continuous from one wall to the other, provided the reinforcing is designed for double the loading. Reinforcement may overlap, provided the reinforcement from one wall does not contact the reinforcement from the other wall. Reinforcement overlaps of more than 3 feet are generally not desirable due to the increased cost of materials. Preapproved proprietary wall systems may be used for back-to-back MSE walls provided they meet the height, height/width ratio and overlap requirements specified herein.

15.4.2.3 Walls on Slopes

Standard Plan walls founded on slopes shall meet the requirements in the Standard Plans. All other walls shall have a near horizontal bench at the wall face at least 4 feet wide to provide access for maintenance. Bearing resistance for footings in slopes and overall stability requirements in the AASHTO LRFD Bridge Design Specifications shall be met (including proprietary walls designed using the AASHTO Standard Specifications for Highway Bridges, 2002).

15.4.2.4 MSE Wall Supported Abutments

MSE walls directly supporting spread footing bridge abutments shall be 20 feet or less in total height. Abutment spread footing service loads should not exceed 3.0 TSF. Proprietary MSE walls supporting abutments shall not be considered preapproved, and shall not be used beyond the limits described herein unless approved by the State Geotechnical Engineer and the Bridge Design Engineer. The front edge of the abutment footing shall be 2 feet or more from the back of the MSE facing units. There shall be at least 5 feet vertical clearance between the MSE facing units and the bottom of the superstructure, and 5 feet horizontal clearance between the back of the MSE facing units and face of the abutment wall to provide access for bridge inspection. Fall protection shall be installed as necessary. These MSE abutment criteria are also applicable to proprietary walls designed using the AASHTO Standard Specifications for Highway Bridges (**2002**).

The bearing resistance for the footing supported by the MSE wall is a function of the soil reinforcement density in addition to the shear strength of the soil. If designing the wall using LRFD, two cases should be evaluated to size the footing for bearing resistance for the strength limit state, as two sets of load factors are applicable:

- The load factors applicable to the structure loads applied to the footing, such as DC, DW, EH, LL, etc.
- The load factor applicable to the distribution of surcharge loads through the soil, ES.

When ES is used to factor the load applied to the soil to evaluate bearing, the structure loads and live load applied to the footing should be unfactored. When ES is not used to factor the load applied to the soil to evaluate bearing, the structure loads and live load applied to the footing should be factored using DC, DW, EH, LL, etc. The wall should be designed for both cases, and the case that results in the greatest amount of soil reinforcement should be used for the final strength limit state design.

15.4.2.5 Minimum Embedment

All walls and abutments should meet the minimum embedment criteria in AASHTO. The final embedment depth required shall be based on geotechnical bearing and stability requirements provided in the AASHTO LRFD specifications, as determined by the geotechnical designer. Walls that have a sloping ground line at the face of wall may need to have a sloping or stepped foundation to optimize the wall embedment. Sloping foundations (i.e., not stepped) shall be 4H:1V or flatter. Stepped foundations shall be 1.5H:1V or flatter determined by a line through the corners of the steps. The maximum feasible slope of stepped foundations for walls is controlled by the maximum acceptable stable slope for the soil in which the wall footing is placed. Concrete leveling pads constructed for MSE walls shall be sloped at 4H:1V or flatter or stepped at 1.5H:1V or flatter determined by a line through the corners of the steps. As MSE wall facing units are typically rectangular shapes, stepped leveling pads are preferred. These embedment criteria are also applicable to proprietary walls designed using the AASHTO Standard Specifications for Highway Bridges (**2002**).

15.4.2.6 Serviceability Requirements

Walls shall be designed to structurally withstand the effects of total and differential settlement estimated for the project site, both longitudinally and in cross-section, as prescribed in the AASHTO LRFD Specifications. In addition to the requirements for serviceability provided above, the following criteria (**Tables 15-2, 15-3, and 15-4**) shall be used to establish acceptable settlement criteria (including proprietary walls designed using the AASHTO Standard Specifications for Highway Bridges, 2002):

Total Settlement	Differential Settlement Over 100 ft	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and Construct
1 in < $\Delta H \leq 2.5$ in	0.75 in < $\Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 15-2 Settlement criteria for Reinforced Concrete Walls, Nongravity Cantilever Walls, Anchored/Braced Walls, and MSE Walls with Full Height Precast Concrete Panels (soil is place directly against panel).

Total Settlement	Differential Settlement Over 100 ft	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and Construct
2 in < $\Delta H \leq 4$ in	1.5 in < $\Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 15-3 Settlement criteria for MSE Walls with Modular (segmental) Block Facings, Prefabricated Modular Walls, and Rock Walls.

Total Settlement	Differential Settlement Over 50 ft	Action
$\Delta H \leq 4$ in	$\Delta H_{50} \leq 3$ in	Design and Construct
4 in < $\Delta H \leq 12$ in	3 in < $\Delta H_{50} \leq 9$ in	Ensure structure can tolerate settlement
$\Delta H > 12$ in	$\Delta H_{50} > 9$ in	Obtain Approval[1] prior to proceeding with design and Construction

[1]Approval of WSDOT State Geotechnical Engineer and WSDOT Bridge Design Engineer required.

Table 15-4 Settlement criteria for MSE Walls with Flexible Facings and Reinforced Slopes.

For MSE walls with precast panel facings up to 75 ft^2 in area, limiting differential settlements shall be as defined in the AASHTO LRFD Specifications, Article C11.10.4.1.

Note that more stringent tolerances may be necessary to meet aesthetic requirements for the walls.

15.4.2.7 Active, Passive, At-rest Earth Pressures

The geotechnical designer shall assess soil conditions and shall develop earth pressure diagrams for all walls except standard plan walls in accordance with the AASHTO LRFD Bridge Design Specifications. Earth pressures may be based on either Coulomb or Rankine theories. The type of earth pressure used for design depends on the ability of the wall to yield in response to the earth loads. For walls that free to translate or rotate (i.e., flexible walls), active pressures shall be used in the retained soil. Flexible walls are further defined as being able to displace laterally at least 0.001H, where H is the height of the wall. Standard concrete walls, MSE walls, soil nail walls, soldier pile walls and anchored walls are generally considered as flexible retaining walls. Non-yielding walls shall use at-rest earth pressure parameters. Nonyielding walls include, for example, integral abutment walls, wall corners, cut and cover tunnel walls, and braced walls (i.e., walls that are cross-braced to another wall or structure. Where bridge wing and curtain walls join the bridge abutment, at rest earth pressures should be used. At distances away from the bridge abutment equal to or greater than the height of the abutment wall, active earth pressures may be used. This assumes that at such distances away from the bridge abutment, the wing or curtain wall can deflect enough to allow active conditions to develop.

If external bracing is used, active pressure may be used for design. For walls used to stabilize landslides, the applied earth pressure acting on the wall shall be estimated from limit equilibrium stability analysis of the slide and wall (external and global stability only). The earth pressure force shall be the force necessary to achieve stability in the slope, which may exceed at-rest or passive pressure.

15.4.2.8 Surcharge Loads

Article 3.11.6 in the AASHTO LRFD Bridge Design Specifications shall be used for surcharge loads acting on all retaining walls and abutments.

15.4.2.9 Seismic Earth Pressures

For all walls and abutments, the Mononobe-Okabe method described in the AASHTO LRFD Bridge Design Specifications, Chapter 11 and Appendix A11.1.1.1, shall be used. In addition, for this approach it is assumed that the wall backfill is completely drained and cohesionless (i.e. not susceptible to liquefaction).

Walls and abutments that are free to translate or move during a seismic event (see **Section 15.4.2.6**) may use a reduced horizontal acceleration k_h of approximately ½ peak ground acceleration or as specifically calculated in Article 11.6.5 of the AASHTO LRFD Bridge Design Specifications in the Mononobe-Okabe method. Vertical acceleration, k_v, should be set equal to 0.

Walls and abutments that are not free to translate or move during a seismic event (see **Section 15.4.2.6**) shall use a horizontal acceleration of 1.5 times peak ground acceleration. Vertical acceleration shall be set equal to 0.

The current AASHTO specifications are not consistent regarding the location of the resultant, nor are they consistent regarding the separation of the static earth pressure from the seismic earth pressure (i.e., the use of ΔK_{ae} to represent the seismic portion of the earth pressure versus the use of K_{ae} to represent the total of the seismic and static earth pressure). Until this issue is resolved, the following policy shall be implemented regarding seismic earth pressure calculation:

- The seismic "component" of the Mononobe-Okabe earth pressure may be separated from the static earth pressure acting on the wall as shown in Article 11.10.7.1 in the AASHTO LRFD Bridge Design Specifications. If this is done, the seismic component, ΔKae, shall be calculated as Kae – Ka for walls that are free to move and develop active earth pressure conditions, and as Kae – K_0 for walls that are not free to move (i.e., at rest earth pressure conditions prevail, and Kae is calculated using a horizontal acceleration of 1.5 times the peak ground acceleration). Note that in this case, to complete the seismic design of the wall, the static earth pressure resulting from Ka or K_0 **must be added** to the seismic component of the earth pressure resulting from ΔKae to obtain the total earth pressure acting in the extreme event limit state. The load factor for EQ in Section 3 of the AASHTO LRFD Bridge Design Specifications shall be applied to the static and seismic earth pressure loads, since in Mononobe-Okabe earth pressure analysis, a total static plus seismic earth pressure is calculated as one force initially, and then separated into the static and seismic components as a second step.
- The resultant force of the Mononobe-Okabe earth pressure distribution, as represented by ΔKae should be applied at 0.6H from the bottom of the pressure distribution. Note that the distribution is an inverted trapezoid if the resultant is applied at 0.6H, with the pressure at the top of the distribution equal to 0.8ΔKaeγH, and the pressure at the bottom equal to 0.2ΔKaeγH.
- If the seismic earth pressure force is calculated and distributed as a single force as specified in Appendix A11.1.1.1 of the AASHTO LRFD Bridge Design Specifications, the combined earth pressure force shall be applied at 0.5H from the bottom of the pressure distribution, resulting in a uniform pressure distribution in which the pressure is equal to 0.5 KaeγH. Note that since this uniform pressure distribution includes both the static and seismic component of lateral earth pressure, this uniform earth pressure **must not be added** to the earth pressure resulting from Ka or K_0.
- For all walls, the pressure distribution should be applied from the bottom of wall to the top of wall except cantilever walls, anchored walls, or braced walls. For these walls, the pressure should be applied from the top of wall to the elevation of finished ground line at the face of wall.

The Mononobe-Okabe seismic earth pressure theory was developed for a single layer cohesionless soil with no water present. For most gravity walls, this assumption is applicable in most cases. However, for cut walls such as anchored walls or non-gravity cantilever walls, it is possible and even likely that these assumptions may not be applicable. In such cases where these assumptions are not fully applicable, a weighted average (weighted based on the thickness of each layer) of the soil properties (e.g., effective stress ϕ and γ) should be used to calculate K_{ae}. Only the soil above the dredge line or finished grade in front of the wall should be included in the weighted average. If water behind the wall cannot be fully drained, the lateral pressure due to the difference in head must be added to the pressure resulting from K_{ae} to obtain the total lateral force acting in the seismic limit state (note K_{ae} includes the total of seismic and active earth pressure, as described previously). If cohesive soils are present behind the wall, the residual drained friction angle rather than the peak friction angle (see **WSDOT GDM Chapter 5**) should be used to determine the seismic lateral earth pressure.

Note also that the slope of the active failure plane flattens as the earthquake acceleration increases. For anchored walls, the anchors should be located behind the active failure wedge. The methodology provided in FHWA Geotechnical Engineering Circular No. 4 (**Sabatini, et al., 1999**) should be used to locate the active failure plane for the purpose of anchored zone location for anchored walls.

Since the load factor used for the seismic lateral earth pressure for EQ is currently 1.0, to obtain the same level of safety for sliding and bearing obtained from the AASHTO Standard Specification design requirements, a resistance factor of slightly less than 1.0 is required. For sliding and bearing resistance during seismic loading, a resistance factor of 0.9 should be used.

The seismic design criteria provided in this section are also applicable to proprietary walls designed using the AASHTO Standard Specifications for Highway Bridges (**2002**).

15.4.2.10 Liquefaction

Under extreme event loading, liquefaction and lateral spreading may occur. The geotechnical designer shall assess liquefaction and lateral spreading for the site and identify these geologic hazards. Design to assess and to mitigate these geologic hazards shall be conducted in accordance with the provisions in **WSDOT GDM Chapter 6**.

15.4.2.11 Overall Stability

All retaining walls and reinforced slopes shall have a resistance factor for overall stability of 0.75 (i.e., a safety factor of 1.3). All abutments and those retaining walls and reinforced slopes deemed critical shall have a resistance factor of 0.65 (i.e., a safety factor of 1.5). Critical walls and slopes are those that support important structures like bridges and other retaining walls. Critical walls and slopes would also be those whose failure would result in a life threatening safety hazard for the public, or whose failure and subsequent replacement or repair would be an intolerable financial burden to the citizens of Washington State.

Stability shall be assessed using limiting equilibrium methods in accordance with **WSDOT GDM Chapter 7**.

15.4.2.12 Wall Drainage

Drainage should be provided for all walls. In instances where wall drainage cannot be provided, the hydrostatic pressure from the water shall be included in the design of the wall. In general, wall drainage shall be in accordance with the Standard Plans, General Special Provisions, and the WSDOT Design Manual. Figure 1130-2 in the design manual shall be used for drain details and drain placement for all walls not covered by WSDOT Standard Plan D-4 except as follows:

- Gabion walls and rock walls are generally considered permeable and do not typically require wall drains, provided construction geotextile is placed against the native soil or fill.
- Soil nail walls shall use composite drainage material centered between each column of nails. The drainage material shall be connected to weep holes using a drain gate or shall be wrapped around an underdrain.
- Cantilever and Anchored wall systems using lagging shall have composite drainage material attached to the lagging face prior to casting the permanent facing. Walls without facing or walls using precast panels are not required to use composite drainage material provided the water can pass through the lagging unhindered.

15.4.2.13 Utilities

Walls that have or may have future utilities in the backfill should minimize the use of soil reinforcement. MSE, soil nail, and anchored walls commonly have conflicts with utilities and should not be used when utilities must remain in the reinforced soil zone unless there is no other wall option. Utilities that are encapsulated by wall reinforcement may not be accessible for replacement or maintenance. Utility agreements should specifically address future access if wall reinforcing will affect access.

15.4.2.14 Guardrail and Barrier

Guardrail and barrier shall meet the requirements of the State Design Manual, Bridge Design Manual, Standard Plans, and the AASHTO LRFD Bridge Design Specifications. In no case shall guardrail be placed through MSE wall or reinforced slope soil reinforcement closer than 3 ft from the back of the wall facing elements. Furthermore, the guard rail posts shall be installed through the soil reinforcement in a manner that prevents ripping and distortion of the soil reinforcement, and the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.

15.5 Specific Design Requirements

15.5.1 Abutments and Standard Plan Walls

Abutment foundations shall be designed in accordance with **WSDOT GDM Chapter 8**. Abutment walls, wingwalls, and curtain walls shall be designed in accordance with AASHTO LRFD Bridge Design Specifications. Abutments that are backfilled prior to constructing the superstructure shall be designed using active earth pressures. Active earth pressures shall be used for abutments that are backfilled after construction of the superstructure, if the abutment can move sufficiently to develop active pressures. If the abutment is restrained, at-rest earth pressure shall be used. Abutments that are “U” shaped or that have curtain/wing walls should be designed to resist at-rest pressures in the corners, as the walls are constrained (see **WSDOT GDM Section 15.4.2.7**).

For standard plan walls, the internal stability design and the external stability design for overturning and sliding stability have already been completed. The geotechnical designer shall assess overall slope stability, soil bearing resistance, and settlement for each standard plan wall location. Since these Standard Plan walls have been designed using Load Factor Design per the AASHTO Standard Specifications for Highway Bridges (2002), geotechnical safety factors consistent with the AASHTO Standard Specifications shall be used for Standard Plan walls until such time that they have been updated to use LRFD methodology.

15.5.2 Nongravity Cantilever and Anchored Walls

WSDOT typically does not utilize sheet pile walls for permanent applications, except at Washington State Ferries (WSF) facilities. Sheet pile walls may be used at WSF facilities but shall not be used elsewhere without approval of the WSDOT Bridge Design Engineer. Sheet pile walls utilized for shoring or cofferdams shall be the responsibility of the Contractor and shall be approved on construction, unless the construction contract special provisions or plans state otherwise.

Permanent soldier piles for soldier pile and anchored walls should be installed in drilled holes. Impact or vibratory methods may be used to install temporary soldier piles, but installation in drilled holes is preferred.

Nongravity and Anchored walls shall be designed using the latest edition of the AASHTO LRFD Bridge Design Specifications. Key geotechnical design requirements for these types of walls are found in Sections 3 and 11 of the AASHTO LRFD specifications. Instead of the resistance factor for passive resistance of the vertical wall elements provided in the AASHTO LRFD specifications, a resistance factor for passive resistance of 0.75 shall be used.

15.5.2.1 Nongravity Cantilever Walls

The exposed height of nongravity cantilever walls is generally controlled by acceptable deflections at the top of wall. In "good" soils, cantilever walls are generally 12 to 15 feet or less in height. Greater exposed heights can be achieved with increased section modulus or the use of secant/tangent piles. Nongravity cantilever walls using a single row of ground anchors or deadmen anchors shall be considered an anchored wall.

In general, the drilled hole for the soldier piles for nongravity cantilever walls will be filled with a relatively low strength flowable material such as controlled density fill (CDF), provided that water is not present in the drilled hole. Since CDF has a relatively low cement content, the cementitious material in the CDF has a tendency to wash out when placed through water. If the CDF becomes too weak because of this, the design assumption that the full width of the drilled hole, rather than the width of the soldier pile by itself, governs the development of the passive resistance in front of the wall will become invalid. The presence of groundwater will affect the choice of material specified by the structural designer to backfill the soldier pile holes, e.g., CDF if the hole is not wet, or higher strength concrete designed for tremie applications. Therefore, it is important that the geotechnical designer identify the potential for ground water in the drilled holes during design, as the geotechnical stability of a nongravity cantilever soldier pile wall is governed by the passive resistance available in front of the wall.

If the wall is being used to stabilize a deep seated landslide, in general, it should be assumed that full strength concrete will be used to backfill the soldier pile holes, as the shearing resistance of the concrete will be used to help resist the lateral forces caused by the landslide.

15.5.2.2 Anchored/Braced Walls

Anchored/braced walls generally consist of a vertical structural elements such as soldier piles or drilled shafts and lateral anchorage elements placed beside or through the vertical structural elements. Design of these walls shall be in accordance with the AASHTO LRFD Bridge Design Specifications.

In general, the drilled hole for the soldier piles for anchored/braced walls will be filled with a relatively low strength flowable material such as controlled density fill (CDF). For anchored walls, the passive resistance in front of the wall toe is not as critical for wall stability as is the case for nongravity cantilever walls. For anchored walls, resistance at the wall toe to prevent "kickout" is primarily a function of the structural bending resistance of the soldier pile itself. Therefore, it is not as critical that the CDF maintain its full shear strength during and after placement if the hole is wet. For anchored/braced walls, the only time full strength concrete would be used to fill the soldier pile holes in the buried portion of the wall is when the anchors are steeply dipping, resulting in relatively high vertical loads, or for the case when additional shear strength is needed to resist high lateral kickout loads resulting from deep seated landslides. In the case of walls used to stabilize deep seated landslides, the geotechnical designer must clearly indicate to the structural designer whether or not the shear resistance of the soldier pile and cementitious backfill material (i.e., full strength concrete) must be considered as part of the resistance needed to help stabilize the landslide.

15.5.2.3 Permanent Ground Anchors

The geotechnical designer shall define the no-load zone for anchors in accordance with the AASHTO LRFD Bridge Design Specifications. If the ground anchors are installed through landslide material or material that could potentially be unstable, the no load zone shall include the entire unstable zone as defined by the actual or potential failure surface plus 5 ft minimum. The contract documents should require the drill hole in the no load zone to be backfilled with a non-structural filler. Contractors may request to fill the drill hole in the no load zone with grout prior to testing and acceptance of the anchor. This is usually acceptable provided bond breakers are present on the strands, the anchor unbonded length is increased by 8 feet minimum, and the grout in the unbonded zone is not placed by pressure grouting methods.

The geotechnical designer shall determine the factored anchor pullout resistance that can be reasonably used in the structural design given the soil conditions. The ground anchors used on the projects shall be designed by the Contractor. Compression anchors (see **Sabatini, et al., 1999**) may be used, but conventional anchors are preferred by WSDOT.

The geotechnical designer shall estimate the nominal anchor bond stress (τ_n) for the soil conditions and common anchor grouting methods. AASHTO LRFD Bridge Design Specifications and the FHWA publications listed at the beginning of this chapter provide guidance on acceptable values to use for various types of soil and rock. The geotechnical designer shall then apply a resistance factor to the nominal bond stress to determine a feasible factored pullout resistance (FPR) for anchors to be used in the wall. In general, a 5-inch diameter low pressure grouted anchor with a bond length of 15 to 30 feet should be assumed when estimating the feasible anchor resistance. FHWA research has indicated that anchor bond lengths greater than 40 feet are not fully effective. Anchor bond lengths greater than 50 feet shall be approved by the State Geotechnical Engineer.

The structural designer shall use the factored pullout resistance to determine the number of anchors required to resist the factored loads. The structural designer shall also use this value in the contract documents as the required anchor resistance that Contractor needs to achieve. The Contractor will design the anchor bond zone to provide the specified resistance. The Contractor will be responsible for determining the actual length of the bond zone, hole diameter, drilling methods, and grouting method used for the anchors.

All ground anchors shall be proof tested, except for anchors that are subjected to performance tests. A minimum of 5 percent of the wall's anchors shall be performance tested. For ground anchors in clays, or other soils that are known to be potentially problematic, especially with regard to creep, at least one verification test shall be performed in each soil type within the anchor zone. Past WSDOT practice has been to perform verification tests at two times the design load with proof and performance tests done to 1.5 times the design load. National practice has been to test to 1.33 times the design load for proof and performance tests. Historically, WSDOT has utilized a higher safety factor in its anchored wall designs (FS=1.5) principally due to past performance with anchors constructed in Seattle Clay. For anchors that are installed in Seattle Clay, other similar formations, or clays in general, the level of safety obtained in past WSDOT practice shall continue to be used (i.e., FS = 1.5). For anchors in other soils (e.g., sands, gravels, glacial tills, etc.), the level of safety obtained when applying the national practice (i.e., FS = 1.33) should be used.

The AASHTO LRFD Bridge Design Specifications specifically addresses anchor testing. However, to be consistent with previous WSDOT practice, verification tests, if conducted, shall be performed to 1.5 times the factored design load (FDL) for the anchor. Proof and performance tests shall be performed to 1.15 times the factored design load (FDL) for anchors installed in clays, and to 1.00 times the factored design load (FDAL) for anchors in other soils and rock. The geotechnical designer should make the decision during design as to whether or not a higher test load is required for anchors in a portion of, or all of, the wall due to the presence of clays or other problematic soils.

The following shall be used for verification tests:

Load	Hold Time
AL	1 Min.
0.25FDL	10 Min.
0.50FDL	10 Min.
0.75FDL	10 Min.
1.00FDL	10 Min.
1.15FDL	**60 Min.**
1.25FDL	10 Min.
1.50FDL	10 Min.
AL	1 Min.

AL is the alignment load. The test load shall be applied in increments of 25 percent of the design load. Each load increment shall be held for at least 10 minutes. Measurement of anchor movement shall be obtained at each load increment. The load-hold period shall start as soon as the test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, and 60 minutes.

The following shall be used for proof tests, for anchors in clay or other creep susceptible or otherwise problematic soils or rock:

Load	Hold Time
AL	1 Min.
0.25FDL	1 Min.
0.50FDL	1 Min.
0.75FDL	1 Min.
1.00FDL	1 Min.
1.15FDL	**10 Min.**
AL	1 Min.

The following shall be used for proof tests, for anchors in sands, gravels, glacial tills, rock, or other materials where creep is not likely to be a significant issue:

Load	Hold Time
AL	1 Min.
0.25FDL	1 Min.
0.50FDL	1 Min.
0.75FDL	1 Min.
1.00FDL	**10 Min.**
AL	1 Min.

The maximum test load in a proof test shall be held for ten minutes, and shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between one minute and ten minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the anchor movements shall be recorded at 15, 20, 25, 30, 45, and 60 minutes.

Performance tests cycle the load applied to the anchor. Between load cycles, the anchor is returned to the alignment load (AL) before beginning the next load cycle. The following shall be used for performance tests:

Cycle 1	Cycle 2	Cycle 3	Cycle 4	Cycle 5*	Cycle 6
AL	AL	AL	AL	AL	AL
0.25FDL	0.25FDL	0.25FDL	0.25FDL	0.25FDL	Lock-off
	0.50FDL	0.50FDL	0.50FDL	0.50FDL	
		0.75FDL	0.75FDL	0.75FDL	
			1.00FDL	1.00FDL	
				1.15FDL	

*The fifth cycle shall be conducted if the anchor is installed in clay or other problematic soils. Otherwise, the load hold is conducted at 1.00FDL and the fifth cycle is eliminated.

The load shall be raised from one increment to another immediately after a deflection reading. The maximum test load in a performance test shall be held for ten minutes. If the anchor movement between one minute and ten minutes exceeds 0.04 inches, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the anchor movements shall be recorded at 15, 20, 25, 30, 45, and 60 minutes. After the final load hold, the anchor shall be unstressed to the alignment load then jacked to the lock-off load.

The structural designer should specify the lock-off load in the contract. Past WSDOT practice has been to lock-off at 80% of the anchor design load. Because the factored design load for the anchor is higher than the "design load" used in past practice, locking off at 80% would result in higher tendon loads. To match previous practice, the lock-off load for all permanent ground anchors shall be 60% of the factored design load for the anchor.

Since the contractor designs and installs the anchor, the contract documents should require the following:

1. Factored design load (FDL) shall not exceed 60% of the specified minimum tensile strength (SMTS) for the anchor.
2. Lock off shall not exceed 70% of the specified minimum tensile strength for the anchor.
3. Test loads shall not exceed 80% of the specified minimum tensile strength for the anchor.
4. All anchors shall be double corrosion protected (encapsulated). Epoxy coated or bare strands shall not be used unless the wall is temporary.
5. Ground anchor installation angle should be 15 to 30 degrees from horizontal, but may be as steep as 45 degrees to install anchors in competent materials or below failure planes.

The geotechnical designer and the structural designer should develop the construction plans and special provisions to ensure that the contractor complies with these requirements.

15.5.2.4 Deadmen

The geotechnical designer shall develop earth pressures and passive resistance for deadmen in accordance with AASHTO LRFD Bridge Design Specifications. Deadmen shall be located in accordance with Figure 20 from NAVFAC DM-7.2, Foundations and Earth Structures, May 1982 (reproduced below for convenience in **Figure 15-2**).

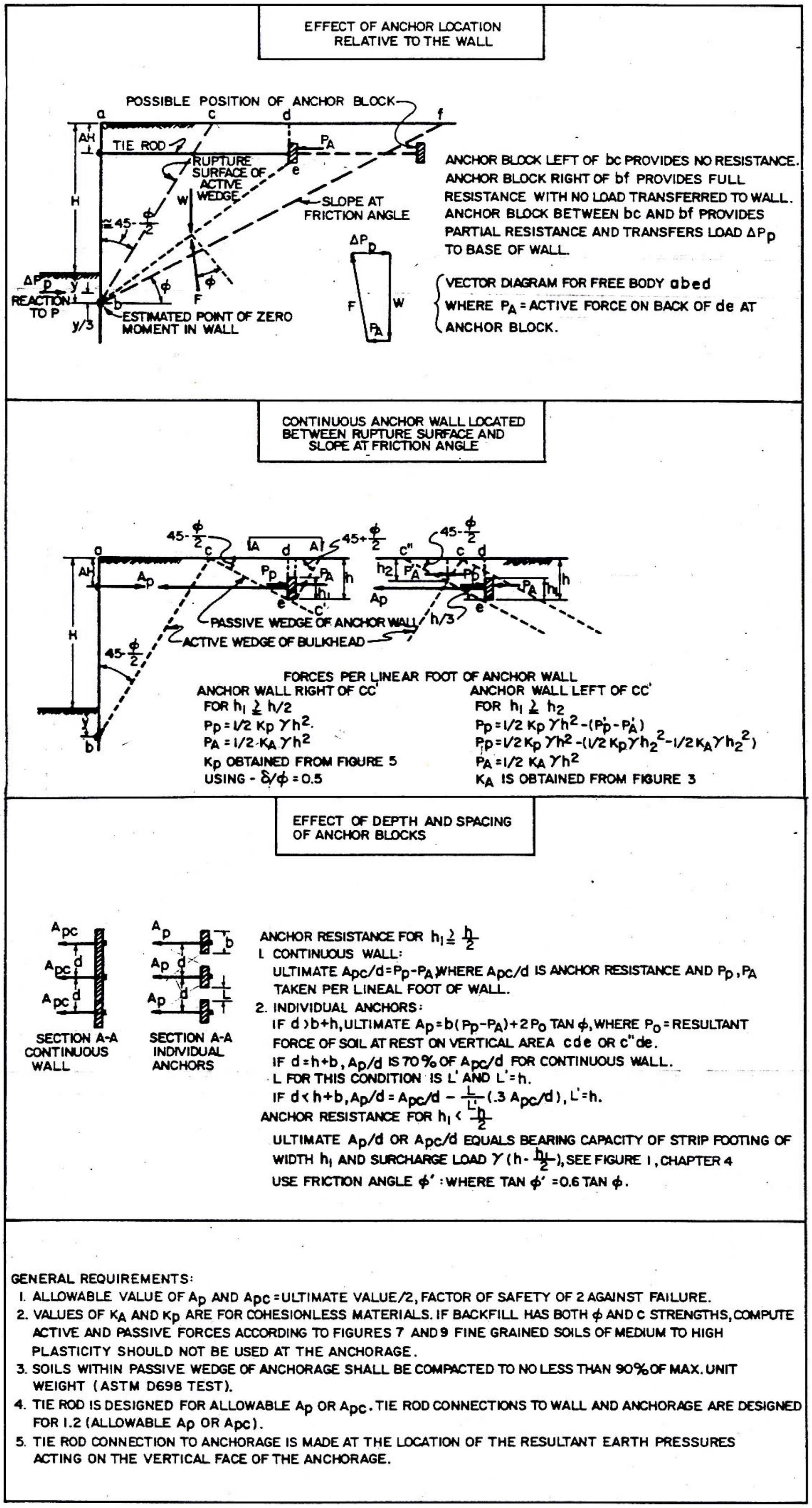

Figure 15-2 Deadman anchor design (after **NAVFAC, 1982**).

15.5.3 Mechanically Stabilized Earth Walls

Preapproved wall systems shall be 33 feet or less in total height. Specific proprietary wall systems may have more stringent height limitations. Greater wall heights may be used, but a special design (i.e., not preapproved) will be required. Wall design shall be in accordance with the AASHTO LRFD Bridge Design Specifications, except as noted below regarding the use of the K-Stiffness Method for internal stability design. As noted previously, WSDOT will allow a grace period for the proprietary wall systems preapproved on or before December 1, 2004, and that have remained in approved status until the present, regarding the implementation of the LRFD approach. In those cases, the AASHTO Standard Specifications for Highway Bridges (**2002**), as modified in the WSDOT GDM, may be used for the design of those systems until such time that WSDOT decides to end the grace period.

For walls with a traffic barrier, design of the traffic barrier and the distribution of the applied impact load to the wall top shall be as described in the AASHTO Standard Specifications for Highway Bridges **(2002)**, Article 5.8.12.2, for both AASHTO Standard Specification wall designs and AASHTO LRFD Specification designs.

15.5.3.1 Internal Stability Using K-Stiffness Method

The K-Stiffness Method, as described by **Allen and Bathurst (2003)**, may be used as an alternative to the Simplified Method provided in the AASHTO LRFD Bridge Design Specifications (Sections 3 and 11) to design the internal stability for walls up to 25 ft in height that are not directly supporting other structures and that are not in high settlement areas. Use of the K-Stiffness Method for greater wall heights, in locations where settlement is anticipated to be greater than 6 inches, or for walls that support other structures shall be considered experimental, will require special monitoring of performance, and the approval of the State Geotechnical Engineer. The AASHTO LRFD Bridge Design Specifications are applicable, as well as the traffic barrier design provisions in the WSDOT LRFD BDM, except as modified in the provisions that follow.

15.5.3.1.1 K-Stiffness Method Loads and Load Factors

In addition to the load factors provided in Section 3.4.1 of the AASHTO LRFD specifications, the load factors provided in **Table 15-2** shall be used as minimum values for the K-Stiffness Method. The load factor γ_p to be applied to maximum load carried by the reinforcement T_{max} due to the weight of the backfill for reinforcement strength, connection strength, and pullout calculations shall be EV, for vertical earth pressure.

Type of Load	Load Factor	
	Maximum	Minimum
EV: Vertical Earth Pressure:		
MSE Wall soil reinforcement loads (K-Stiffness Method, steel strips and grids)	1.55	N/A
MSE Wall soil reinforcement/facing connection loads (K-Stiffness Method, steel grids attached to rigid facings)	1.80	N/A
MSE Wall soil reinforcement loads (K-Stiffness Method, geosynthetics)	1.60	N/A
MSE Wall soil reinforcement/facing connection loads (K-Stiffness Method, geosynthetics)	1.85	N/A

Table 15-2 Load Factors for Permanent Loads for internal stability of MSE walls designed using the K-Stiffness Method, γ_p.

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The calculation method for T_{max} is empirically derived, based on reinforcement strain measurements, converted to load based on the reinforcement stiffness, from full scale walls at working stress conditions (see **Allen and Bathurst, 2003**). Research by **Allen and Bathurst (2003)** indicates that the working loads measured in MSE wall reinforcement remain relatively constant throughout the wall life, provided the wall is designed for a stable condition, and that the load statistics remain constant up to the point that the wall begins to fail. Therefore, the load factors for MSE wall reinforcement loads provided in **Table 15-2** can be considered valid for a strength or extreme event limit state.

The load factors provided in **Table 15-2** were determined assuming that the appropriate mean soil friction angle is used for design. In practice, since the specific source of material for wall backfill is typically not available at the time of design, presumptive design parameters based on previous experience with the material that is typically supplied to meet the backfill material specification (e.g., Gravel Borrow per the WSDOT Standard Specifications for construction) are used (see **WSDOT GDM Chapter 5**). It is likely that these presumptive design parameters are lower bound conservative values for the backfill material specification selected. Triaxial or direct shear soil friction angles should be used with the Simplified Method provided in the AASHTO LRFD Specifications, to be consistent with the current specifications and empirical derivation for the Simplified Method, whereas plane strain soil friction angles should be used with the K-Stiffness Method, to be consistent with the empirical derivation and calibration for that method. The following equations maybe used to make an approximate estimate of the plane strain soil friction angle based on triaxial or direct shear test results.

For triaxial test data (**Lade and Lee, 1976**):

$$\phi_{ps} = 1.5\phi_{tx} - 17 \qquad (15\text{-}1)$$

For direct shear test data (based on interpretation of data presented by **Bolton (1986) and Jewell and Wroth (1987)**):

$$\phi_{ps} = \tan^{-1}(1.2 \tan \phi_{ds}) \qquad (15\text{-}2)$$

All soil friction angles are in degrees for both equations. Direct shear or triaxial soil friction angles may be used for design using the K-Stiffness Method, if desired, but it should be recognized that doing so could add some conservatism to the resulting load prediction. Note that if presumptive design parameters are based on experience from triaxial or direct shear testing of the backfill, a slight increase in the presumptive soil friction angle based on **equations 15-1 or 15-2** is appropriate to apply.

Other loads appropriate to the load groups and limit states to be considered as specified in the AASHTO LRFD specifications for wall design are applicable when using the K-Stiffness Method for design.

15.5.3.1.2 K-Stiffness Method Resistance Factors

For the service limit state, a resistance factor of 1.0 should be used, except for the evaluation of overall slope stability as prescribed by the AASHTO LRFD specifications (see also **Section 15.4.2.10**). For the strength and extreme event limit states for internal stability using the K-Stiffness Method, the resistance factors provided in **Table 15-3** shall be used as maximum values. These resistance factors were derived using the data provided in **Allen and Bathurst (2003)**. Reliability theory, using the Monte Carlo Method as described in **Allen, et al. (in press)** was applied to statistically characterize the data and to estimate resistance factors. The load factors provided in **Table 15-2** were used for this analysis.

The resistance factors, specified in **Table 15-3** are consistent with the use of select granular backfill in the reinforced zone, homogeneously placed and carefully controlled in the field for conformance with the WSDOT Standard Specifications. The resistance factors provided in **Table 15-3** have been developed with consideration to the redundancy inherent in MSE walls due to the multiple reinforcement layers and the ability of those layers to share load one with another. This is accomplished by using a target reliability index, β, of 2.3 (approximate probability of failure, P_f, of 1 in 100 for static conditions) and a β of 1.65 (Approximate P_f of 1 in 20) for seismic conditions. A β of 3.5 (approximate P_f of 1 in 5,000) is typically used for structural design when redundancy is not considered or not present; see **Allen et al., in press**, for additional discussion on this issue. Because redundancy is already taken into account through the target value of β selected, the factor η for redundancy prescribed in the AASHTO LRFD specifications should be set equal to 1.0. The target value of β used herein for seismic loading is consistent with the overstress allowed in previous practice as described in the AASHTO Standard Specifications for Highway Bridges (**AASHTO 2002**).

Limit State and Reinforcement Type			Resistance Factor
Internal Stability of MSE Walls, K-Stiffness Method			
φ_{rr}	Reinforcement Rupture	Metallic Geosynthetic	0.85 0.80(3)
φ_{sf}	Soil Failure	Metallic Geosynthetic	0.85 1.00(1)
φ_{cr}	Connection rupture	Metallic Geosynthetic	0.85 0.80(3)
φ_{po}	Pullout(2)	Steel ribbed strips (at z $\leq$ 2 m) Steel ribbed strips (at z > 2 m) Steel smooth strips Steel grids Geosynthetic	1.10 1.00 1.00 0.60 0.50
φ_{EQr}	Combined static/ earthquake loading (reinforcement and connector rupture)	Metallic Geosynthetic	1.00 0.95(3)
φ_{EQp}	Combined static/ earthquake loading (pullout) (2)	Steel ribbed strips (at z $\leq$ 2 m) Steel ribbed strips (at z > 2 m) Steel smooth strips Steel grids Geosynthetic	1.25 1.15 1.15 0.75 0.65

(1) If default value for the critical reinforcement strain of 3.0% or less is used for flexible wall facings, and 2.0% or less for stiff wall facings (for a facing stiffness factor of less than 0.9).

(2) Resistance factor values in table for pullout assume that the default values for F* and α provided in Article 11.10.6.3.2 of the AASHTO LRFD Specifications are used and are applicable.

(3) This resistance factor applies if installation damage is not severe (i.e., RFID < 1.7). Severe installation damage is likely if very light weight reinforcement is used. Note that when installation damage is severe, the resistance factor needed for this limit state can drop to approximately 0.15 or less due to greatly increased variability in the reinforcement strength, which is not practical for design.

Table 15-3 Resistance factors for the strength and extreme event limit states for MSE walls designed using the K-Stiffness Method.

15.5.3.1.3 Safety Against Structural Failure (Internal Stability)

Safety against structural failure shall consider all components of the reinforced soil wall, including the soil reinforcement, soil backfill, the facing, and the connection between the facing and the soil reinforcement, evaluating all modes of failure, including pullout and rupture of reinforcement.

A preliminary estimate of the structural size of the stabilized soil mass may be determined on the basis of reinforcement pullout beyond the failure zone, for which resistance is specified in Article 11.10.6.3 of the AASHTO LRFD Bridge Design Specifications.

The load in the reinforcement shall be determined at two critical locations: the zone of maximum stress and the connection with the wall face. Potential for reinforcement rupture and pullout are evaluated at the zone of maximum stress, which is assumed to be located at the boundary between the active zone and the resistant zone in Figure 11.10.2-1 of the AASHTO LRFD Bridge Design Specifications. Potential for reinforcement rupture and pullout are also evaluated at the connection of the reinforcement to the wall facing. The reinforcement shall also be designed to prevent the backfill soil from reaching a failure condition.

Loads carried by the soil reinforcement in mechanically stabilized earth walls are the result of vertical and lateral earth pressures, which exist within the reinforced soil mass, reinforcement extensibility, facing stiffness, wall toe restraint, and the stiffness and strength of the soil backfill within the reinforced soil mass. The soil reinforcement extensibility and material type are major factors in determining reinforcement load. In general, inextensible reinforcements consist of metallic strips, bar mats, or welded wire mats, whereas extensible reinforcements consist of geotextiles or geogrids. Inextensible reinforcements reach their peak strength at strains lower than the strain required for the soil to reach its peak strength. Extensible reinforcements reach their peak strength at strains greater than the strain required for soil to reach its peak strength. Internal stability failure modes include soil reinforcement rupture or failure of the backfill soil (strength or extreme event limit state), and excessive reinforcement elongation under the design load (service limit state). Internal stability is determined by equating the factored tensile load applied to the reinforcement to the factored tensile resistance of the reinforcement, the tensile resistance being governed by reinforcement rupture and pullout. Soil backfill failure is prevented by keeping the soil shear strain below its peak shear strain.

The methods used in historical design practice for calculating the load in the reinforcement to accomplish internal stability design include the Simplified Method, the Coherent Gravity Method, and the FHWA Structure Stiffness Method. All of these methods are empirically derived, relying on limit equilibrium concepts for their formulation, whereas, the K-Stiffness Method, also empirically derived, relies the difference in stiffness of the various wall components to distribute a total lateral earth pressure derived from limit equilibrium concepts to the wall reinforcement layers and the facing. Though all of these methods can be used to evaluate the potential for reinforcement rupture and pullout for the Strength and Extreme Event limit states, only the K-Stiffness Method can be used to directly evaluate the potential for soil backfill failure and to design the wall internally for the service limit state. These other methods used in historical practice indirectly account for soil failure and service limit state conditions based on the successful construction of thousands of structures (i.e., if the other limit states are met, soil failure will be prevented, and the wall will meet serviceability requirements for internal stability).

These MSE wall specifications also assume that inextensible reinforcements are not mixed with extensible reinforcements within the same wall. MSE walls that contain a mixture of inextensible and extensible reinforcements are not recommended.

The design specifications provided herein assume that the wall facing combined with the reinforced backfill acts as a coherent unit to form a gravity retaining structure. The effect of relatively large vertical spacing of reinforcement on this assumption is not well known and a vertical spacing greater than 2.7 FT should not be used without full scale wall data (e.g., reinforcement loads and strains, and overall deflections) which supports the acceptability of larger vertical spacings. **Allen and Bathurst (2003)** do report that based on data from a number of wall case histories, the correlation between vertical spacing and reinforcement load appears to remain linear for vertical spacings ranging from 1 to 5 ft, though the data at vertical spacings greater than 2.7 ft is very limited. However, larger vertical spacings can result in excessive facing deflection, both localized and global, which could in turn cause localized elevated stresses in the facing and its connection to the soil reinforcement.

The factored vertical stress, σ_V, at each reinforcement level shall be:

$$\sigma_V = \gamma_p \gamma_r H + \gamma_p \gamma_f S + \gamma_{LL} q + \gamma_p \Delta\sigma_V \tag{15-3}$$

where:

σ_V = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (KSF)

γ_P = the load factor for vertical earth pressure EV in **Table 8-2**

γ_{LL} = the load factor for live load surcharge per the AASHTO LRFD Specifications

q = live load surcharge (KSF)

H = the total vertical wall height at the wall face (FT)

S = average soil surcharge depth above wall top (FT)

$\Delta\sigma_V$ = vertical stress increase due to concentrated surcharge load above the wall (KSF)

Note that the methods used in historical practice (e.g., the Simplified Method) calculate the vertical stress resulting from gravity forces within the reinforced backfill at each level, resulting in a linearly increasing gravity force with depth and a triangular lateral stress distribution. The K-Stiffness Method instead calculates the maximum gravity force resulting from the gravity forces within the reinforced soil backfill to determine the maximum reinforcement load within the entire wall reinforced backfill, T_{mxmx}, and then adjusts that maximum reinforcement load with depth for each of the layers using a load distribution factor, D_{tmax} to determine T_{max}. This load distribution factor was derived empirically based on a number of full scale wall cases and verified through many numerical analyses (see **Allen and Bathurst, 2003**).

Note that sloping soil surcharges are taken into account through an equivalent uniform surcharge and assuming a level backslope condition. For these calculations, the wall height "H" is referenced from the top of the wall at the wall face to the top of the bearing pad, excluding any copings and appurtenances.

For the K-Stiffness Method, the load in the reinforcements is obtained by multiplying the factored vertical earth pressure by a series of empirical factors which take into account the reinforcement global stiffness for the wall, the facing stiffness, the facing batter, the local stiffness of the reinforcement, the soil strength and stiffness, and how the load is distributed to the reinforcement layers. The maximum factored load in each reinforcement layer shall be determined as follows:

$$\sigma_V = \gamma_p \gamma_r H + \gamma_p \gamma_f S + \gamma_{LL} q + \gamma_p \Delta\sigma_V \tag{15-4}$$

where,

S_V = tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall), in FT

K = is an index lateral earth pressure coefficient for the reinforced backfill, and shall be set equal to K_0 as calculated per Article 3.11.5.2 of the AASHTO LRFD Specifications. K shall be no less than 0.3 for steel reinforced systems.

σ_V = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present, as calculated in **Equation 15-3** (KSF)

D_{tmax} = distribution factor to estimate T_{max} for each layer as a function of its depth below the wall top relative to T_{mxmx} (the maximum value of T_{max} within the wall)

S_{global} = global reinforcement stiffness (KSF)

Φ_g = global stiffness factor

Φ_{local} = local stiffness factor

Φ_{fb} = facing batter factor

Φ_{fs} = facing stiffness factor

$\Delta\sigma_H$ = horizontal stress increase at reinforcement level resulting from a concentrated horizontal sur charge load per Article 11.10.10.1 of the AASHTO LRFD Specifications (KSF)

D_{tmax} shall be determined from **Figure 15-2**.

The global stiffness, S_{global}, considers the stiffness of the entire wall section, and it shall be calculated as follows:

$$S_{global} = \frac{J_{ave}}{(H/n)} = \frac{\sum_{i=1}^{n} J_i}{H} \qquad (15\text{-}5)$$

where J_{ave} is the average stiffness of all the reinforcement layers within the entire wall section on a per FT of wall width basis (KIPS/FT), J_i is the stiffness of an individual reinforcement layer on a per FT of wall width basis (KIPS/FT), H is the total wall height (FT), and n is the number of reinforcement layers within the entire wall section.

$$\Phi_g = 0.25\left(\frac{S_{global}}{p_a}\right)^{0.25} \qquad (15\text{-}6)$$

where, p_a = atmospheric pressure (a constant equal to 2.11 KSF), and the other variables are as defined previously.

The local stiffness considers the stiffness and reinforcement density at a given layer and is calculated as follows:

$$S_{local} = \frac{J}{S_v} \qquad (15\text{-}7)$$

where J is the stiffness of an individual reinforcement layer (KIPS/FT), and S_v is the vertical spacing of the reinforcement layers near a specific layer (FT). The local stiffness factor, Φ_{local}, is then defined as follows:

$$\Phi_{local} = \left(\frac{S_{local}}{S_{global}}\right)^{a} \qquad (15\text{-}8)$$

where a = a coefficient which is also a function of stiffness. Based on observations from the available data, set a = 1.0 for geosynthetic walls and = 0.0 for steel reinforced soil walls.

The wall face batter factor, Φ_{fb}, which accounts for the influence of the reduced soil weight on reinforcement loads, is determined as follows:

$$\Phi_{fb} = \left(\frac{K_{abh}}{K_{avh}}\right)^{d} \qquad (15\text{-}9)$$

where, K_{abh} is the horizontal component of the active earth pressure coefficient accounting for wall face batter, and K_{avh} is the horizontal component of the active earth pressure coefficient assuming that the wall is vertical, and d = a constant coefficient (recommended to be 0.25 to provide the best fit to the empirical data).

K_{abh} and K_{avh} are determined from the Coulomb equation, assuming no wall/soil interface friction and a horizontal backslope (**AASHTO 2004**), as follows:

$$K_{ab} = \frac{\cos^2(\phi + \omega)}{\cos^3\omega\left[1 + \frac{\sin\phi}{\cos\omega}\right]} \qquad (15\text{-}10)$$

where, ϕ= peak soil friction angle (ϕ_{peak}), and ω = wall/slope face inclination (positive in a clockwise direction from the vertical). The wall face batter ω is set equal to 0 to determine K_{av} using **Equation 15-10**. The horizontal component of the active earth pressure coefficient, assuming no wall/soil interface friction, is determined as follows:

$K_{abh} = K_{ab}\cos(\omega)$ (15-11)

Since for a vertical wall, $\omega = 0^o$, $K_{av} = K_{avh}$.

The facing stiffness factor, Φ_{fs}, was empirically derived to account for the significantly reduced reinforcement stresses observed for geosynthetic walls with segmental concrete block and propped panel wall facings. It is not yet known whether this facing stiffness correction is fully applicable to steel reinforced wall systems. On the basis of data available at the time of this report, **Allen and Bathurst (2003)** recommend that this facing stiffness factor be determined as a function of a non-dimensional facing column stiffness parameter F_f:

$$F_f = \frac{1.5H^5}{ELb_w^3 h_{eff}} p_a \qquad (15\text{-}12)$$

and

$$\Phi_{fs} = \eta\left(F_f\right)^{\kappa} \qquad (15\text{-}13)$$

where, b_w is the thickness of the facing column, L is equal to a unit length of wall, H = the total wall face height, E = the modulus of the facing material, h_{eff} is the equivalent height of an un-jointed facing column that is 100% efficient in transmitting moment throughout the facing column, and p_a, used to preserve dimensional consistency, is atmospheric pressure (equal to 2.11 KSF). The dimensionless coefficients η and κ were determined from an empirical regression of the full-scale field wall data to be 0.5 and 0.14, respectively.

Equation 15-12 was developed by treating the facing column as an equivalent uniformly loaded cantilever beam. It is recognized that **Equation 15-12** represents a rather crude model of the stiffness of a retaining wall facing column, considering that the wall toe may not be completely fixed, the facing column often contains joints (i.e., the beam is not continuous), and the beam is attached to the reinforcement at various points. Since this analysis is being used to isolate the contribution of the facing to the load carrying capacity of the wall system, a simplified model that treats the facing as an isolated beam can be used. Once significant deflection occurs in the facing column, the reinforcement is then forced to carry a greater percentage of the load in the wall system. The full-scale wall data was used by **Allen and Bathurst (2003)** to empirically determine the percentage of load carried by these two wall components. Due to these complexities, these equations have been used in this analysis only to set up the form of a parameter that can be used to represent the approximate stiffness of the facing column.

For modular block faced wall systems, due to their great width, h_{eff} can be considered approximately equal to the average height of the facing column between reinforcement layers, and that the blocks between the reinforcement layers behave as if continuous. The blocks are in compression, partially due to self weight and partially due to downdrag forces on the back of the facing (**Bathurst, et al. 2000**), and can effectively transmit moment throughout the height of the column between the reinforcement layers that are placed between the blocks where the reinforcement is connected to the facing. The compressibility of the reinforcement layer placed between the blocks, however, can interfere with the moment transmission between the blocks above and below the reinforcement layer, effectively reducing the stiffness of the facing column. Therefore, h_{eff} should be set equal to the average vertical reinforcement spacing for this type of facing. Incremental panel faced systems are generally thinner (a thickness of approximately 4 to 5.5 inches) and the panel joints tend to behave as a pinned connection. Therefore, h_{eff} should be set equal to the panel height for this type of facing. The stiffness of flexible wall facings is not as straight-forward to estimate. Until more is known, a facing stiffness factor Φ_{fs} of 1.0 should be used for all flexible faced walls (e.g., welded wire facing, geosynthetic wrapped facings, including such walls where a precast or cast-in-place concrete facing is placed on the wall after the wall is built).

The maximum wall height available where facing stiffness effects could be observed was approximately 20 ft. Data from taller stiff faced walls were not available. It is possible that this facing stiffness effect may not be as strong for much taller walls. Therefore, for walls taller than approximately 25 ft, approval for use of the K-Stiffness Method by the State Geotechnical Engineer is required.

Allen and Bathurst (2003) also discovered that the magnitude of the facing stiffness factor may also be a function of the amount of strain the soil reinforcement allows to occur. It appears that once the maximum reinforcement strain in the wall exceeds approximately 2 percent strain, stiff wall facings tend to reach their capacity to restrict larger lateral earth pressures. To accommodate this strain effect on the facing stiffness factor, for stiff faced walls, the facing stiffness factor increases for maximum reinforcement strains above 2 percent. Because of this, it is recommended that stiff faced walls be designed for maximum reinforcement strains of approximately 2% or less, if a facing stiffness factor Φ_{fs} of less than 0.9 is used.

For steel reinforced walls, this facing stiffness effect has not been verified, though preliminary data indicates that facing stiffness does not affect reinforcement load significantly for steel reinforced systems. Therefore, a facing stiffness factor Φ_{fs} of 1.0 shall be used for all steel reinforced MSE wall systems.

D_{tmax} shall be determined as shown in **Figure 15-2**. **Allen and Bathurst (2003)** found that as the reinforcement stiffness increases, the load distribution as a function of depth below the wall top becomes more triangular in shape. D_{tmax} is the ratio of T_{max} in a reinforcement layer to the maximum reinforcement load in the wall, T_{mxmx}. Note that the empirical distributions provided in **Figure 15-3** apply to walls constructed on a firm soil foundation. The distributions that would result for a rock or soft soil foundation may be different from those shown in this figure, and in general will tend to be more triangular in shape as the foundation soils become more compressible. For walls placed on top of sloping ground where the slope is 3H:1V or steeper, D_{tmax} shall remain equal to 1.0 for the entire bottom half of the wall or more for **Figure 15-3 (a and b)**.

The factored tensile load applied to the soil reinforcement connection at the wall face, T_o, shall be equal to the maximum factored reinforcement tension, T_{max}, for all wall systems regardless of facing and reinforcement type.

Live loads shall be positioned for extreme force effect. The provisions of Article 3.11.6 in the AASHTO LRFD Specifications shall apply.

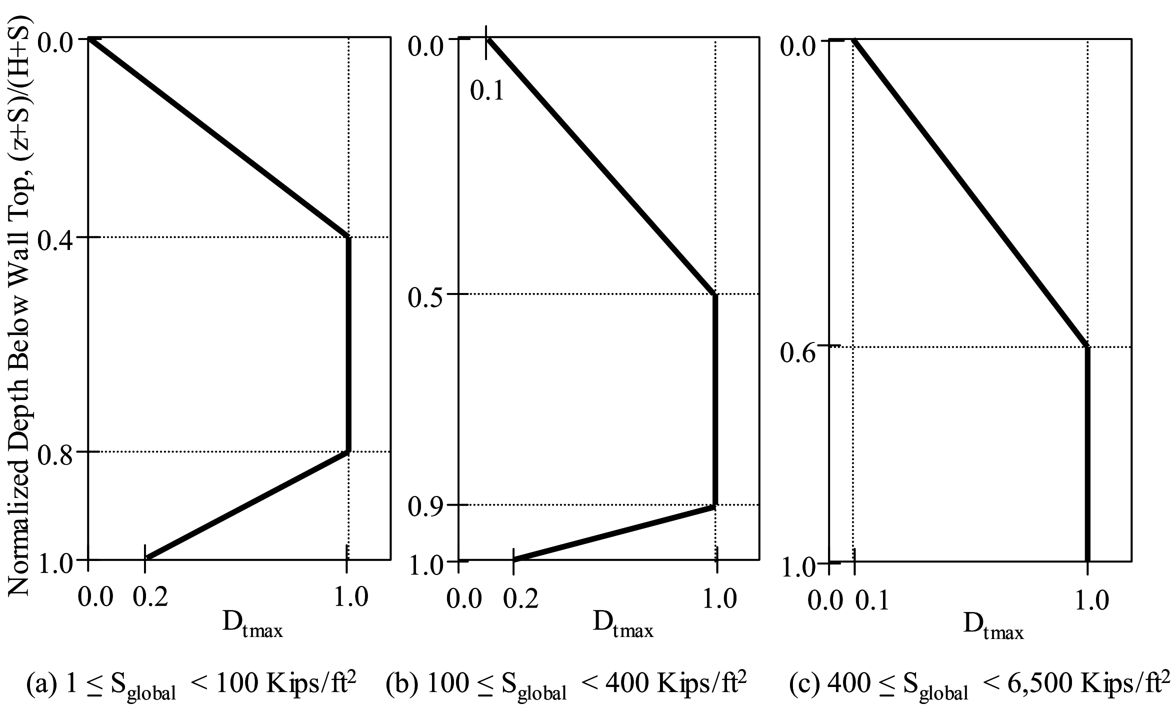

Figure 15-3 D_{tmax} as a function of normalized depth below wall top plus average surcharge depth: (a) generally applies to geosynthetic walls, (b) generally applies to polymer strap walls and extensible or very lightly reinforced steel reinforced systems, and (c) generally applies to steel reinforced systems.

15.5.3.1.4 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Geosynthetic Walls

For geosynthetic walls, four strength limit states (soil failure, reinforcement failure, connection failure, and reinforcement pullout) must be considered for internal reinforcement strength and stiffness design. The design steps, and related considerations, are as follows:

1. Select a trial reinforcement spacing, S_v, and stiffness, J_{EOC}, based on the time required to reach the end of construction (EOC). If the estimated time required to construct the wall is unknown, an assumed construction time of 1,000 hours should be adequate. Note that at this point in the design, it does not matter how one obtains the stiffness. It is simply a value that one must recognize is an EOC stiffness determined through isochronous stiffness curves at a given strain and temperature, and that it represents the stiffness of a continuous reinforcement layer on a per ft of wall width basis. Use the selected stiffness to calculate the trial global stiffness of the wall, S_{global}, using **Equation 15-5**, with J_{EOC} equal to J_i for each layer. Also select a soil friction angle for design (see **WSDOT GDM Section 15.5.3.1.1**). Once the design soil friction angle has been obtained, the lateral earth pressure coefficients needed for determination of T_{max} (Step 4) can be determined (**see WSDOT GDM Section 15.5.3.1.1**). Note that if the reinforcement layer is intended to have a coverage ratio, R_c, of less than 1.0 (i.e., the reinforcement it to be discontinuous), the actual product selected based on the K-Stiffness design must have a stiffness of $J_{EOC}(1/R_c)$.
2. Begin by checking the strength limit state for the backfill soil. The goal is to select a stiffness that is large enough to prevent the soil from reaching a failure condition.
3. Select a target reinforcement strain, ε_{targ}, to prevent the soil from reaching its peak shear strain. The worst condition in this regard is a very strong, high peak friction angle soil, as the peak shear strain for this type of soil will be lower than the peak shear strain obtained from most backfill soils. The results of full-scale wall laboratory testing showed that the reinforcement strain at which the soil begins to exhibit signs of failure is on the order of 3 to 4 percent for high shear strength sands (**Allen and Bathurst, 2003**). This empirical evidence reflects very high shear strength soils and is probably a worst case for design purposes, in that most soils will have larger peak shear strain values than the soils tested in the full-scale walls. A default value for ε_{targ} adequate for granular soils is 3 percent for flexible faced walls, and 2 percent for stiff faced walls if a Φ_{fs} of less than 0.9 is used for design. Lower target strains could also be used, if desired.
4. Calculate the factored load T_{max} for each reinforcement layer (**Equation 15-4**). To determine T_{max}, the facing type, dimensions, and properties must be selected to determine Φ_{fs}. The local stiffness factor Φ_{local} for each layer can be set to 1.0, unless the reinforcement spacing or stiffness within the design wall section is specifically planned to be varied. The global wall stiffness. S_{global}, and global stiffness factor, Φ_g, must be estimated from J_{EOC} determined in Step 1.
5. Estimate the factored strain in the reinforcement at the end of the wall design life, ε_{rein}, using the K-Stiffness Method as follows:

$$\varepsilon_{rein} = \left(\frac{T_{max}}{J_{DL}\varphi_{sf}} \right) \tag{15-14}$$

   where, T_{max} is the factored reinforcement load from Step 4, J_{DL} is the reinforcement layer stiffness at the end of the wall design life (typically 75 years for permanent structures) determined with consideration to the anticipated long-term strain in the reinforcement (i.e., ε_{targ}), ϕ_{sf} is the resistance factor to account for uncertainties in the target strain, and other variables are as defined previously. If a default value of ε_{targ} is used, a resistance factor of 1.0 will be adequate.

6. If ε_{rein} is greater than ε_{targ}, increase the reinforcement layer stiffness J_{EOC} and recalculate T_{max} and ε_{rein}. J_{EOC} will become the stiffness used for specifying the material if the reinforcement layer is continuous (i.e., $R_c = 1$). Note that if the reinforcement layer is intended to have a coverage ratio, R_c, of less than 1.0 (i.e., the reinforcement it to be discontinuous), the actual product selected based on the K-Stiffness design must have a stiffness of $J_{EOC}(1/R_c)$. For final product selection, $J_{EOC}(1/R_c)$ shall be based product specific isochronous creep data obtained in accordance with WSDOT Standard Practice T925 (**WSDOT, 2004**) at the estimated wall construction duration (1,000 hours is an acceptable default time if a specific construction duration of the wall cannot be estimated at time of design) and site temperature. Select the stiffness at the anticipated maximum working strains for the wall, as the stiffness is likely to be strain level dependent. For design purposes, a 2 percent secant stiffness at the wall construction duration time (EOC) is the default strain. If strains of 3 percent are anticipated, determine the stiffness at the higher strain level. If strains of significantly less than 2 percent are anticipated, and a geosynthetic material is being used that is known to have a highly non-linear load-strain curve over the strain range of interest (e.g., some PET geosynthetics), then a stiffness value determined at a lower strain should be obtained. Otherwise, just determine the stiffness at 2 percent strain. This recognizes the difficulties of accurately measuring the stiffness at very low strains. Note that for calculating T_{max}, if multifilament woven geotextiles are to be used as the wall reinforcement, the stiffness values obtained from laboratory isochronous creep data should be increased by 15 percent to account for soil confinement effects. If nonwoven geotextiles are planned to be used as wall reinforcement, J_{EOC} and J_{DL} shall be based on confined in soil isochronous creep data, and use of nonwoven geotextiles shall be subject to the approval of the State Geotechnical Engineer.
7. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term factored rupture strength of the reinforcement is greater than the factored load calculated from the K-Stiffness Method. T_{max} calculated from Step 4 is a good starting point for evaluating this limit state. Note that the global wall stiffness for this calculation is based on the EOC stiffness of the reinforcement, as the reinforcement loads should still be based on EOC conditions, even though the focus of this calculation is at the end of the service life for the wall.
8. Calculate the strength reduction factors RF_{ID}, RF_{CR}, and RF_D for the reinforcement type selected using the approach prescribed in WSDOT Standard Practice T925 (**WSDOT, 2004**). Because the focus of this calculation is to prevent rupture, these factors must be based on reinforcement rupture. Applying a resistance factor to address uncertainty in the reinforcement strength, determine T_{ult}, the ultimate tensile strength of the reinforcement as follows:

$$T_{\max} \leq \frac{T_{ult}\ \varphi_{rr} R_c}{RF_{ID} RF_{CR} RF_D} \tag{15-15}$$

   where, T_{max} is the factored reinforcement load, φ_{rr} is the resistance factor for reinforcement rupture, R_c is the reinforcement coverage ratio, RF_{ID}, RF_{CR}, and RF_D are strength reduction factors for installation damage, creep, and durability, respectively, and the other the variables are as defined previously. The strength reduction factors should be determined using product and site specific data when possible (**AASHTO, 2004; WSDOT, 2004**). T_{ult} is determined from an index wide-width tensile test such as ASTM D4595 or ASTM D6637 and is usually equated to the MARV for the product.
9. Step 8 assumes that a specific reinforcement product will be selected for the wall, as the strength reduction factors for installation damage, creep, and durability are known at the time of design. If the reinforcement properties will be specified generically to allow the contractor or wall supplier to select the specific reinforcement after contract award, use the following equation the long-term design strength of the reinforcement, $T_{aldesign}$:

$$T_{aldesign} = \frac{T_{\max}}{\varphi_{rr} R_c} \quad (15\text{-}16)$$

where T_{max} is the factored reinforcement load from Step 6. The contractor can then select a product with the required $T_{aldesign}$.

10. If the geosynthetic reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. Determine the long-term connection strength ratio CR_{cr} at each reinforcement level, taking into account the available normal force between the facing blocks, if the connection strength is a function of normal force. CR_{cr} is calculated or measured directly per the AASHTO LRFD Specifications.

11. Using the unfactored reinforcement load from Step 6 and an appropriate load factor for the connection load to determine T_{max} (factored) at the connection, determine the adequacy of the long-term reinforcement strength at the connection. Compare the factored connection load at each reinforcement level to the available factored long-term connection strength as follows:

$$T_{\max} \le \varphi_{cr} T_{ac} R_c = \frac{\varphi_{cr} T_{ult} CR_{cr} R_c}{RF_D} \quad (15\text{-}17)$$

where, T_{max} is the factored reinforcement load. Note that for modular block faced walls, the connection test data produced and used for design typically already has been converted to a load per unit width of wall facing – hence, $R_c = 1$. For other types of facing (e.g., precast concrete panels, if discontinuous reinforcement is used (e.g., polymer straps), it is likely that $R_c < 1$ will need to be used in **Equation 15-17**. If the reinforcement strength available is inadequate to provide the needed connection strength as calculated from **Equation 15-17**, decrease the spacing of the reinforcement or increase the reinforcement strength. Then recalculate the global wall stiffness and re-evaluate all previous steps to ensure that the other strength limit states are met. If the strength limit state for reinforcement or connection rupture is controlling the design, increase the reinforcement stiffness and check the adequacy of the design, increasing T_{al} or T_{ult} if necessary.

12. It must be recognized that the strength (T_{ult} and T_{al}) and stiffness (J_{EOC}) determined from the K-Stiffness Method could result in the use of very light weight geosynthetics. In no case shall geosynthetic reinforcement be used that has an RF_{ID} applicable to the anticipated soil backfill gradation and installation conditions anticipated of greater than 1.7, as determined per WSDOT Standard Practice T925 (**WSDOT, 2004**). Furthermore, reinforcement coverage ratios, R_c, of less than 1.0 may be used provided that it can be demonstrated the facing system is fully capable of transmitting forces from un-reinforced segments laterally to adjacent reinforced sections through the moment capacity of the facing elements. For walls with modular concrete block facings, the gap between soil reinforcement sections or strips at a horizontal level shall be limited to a maximum of one block width in accordance with the AASHTO LRFD Specifications, to limit bulging of the facing between reinforcement levels or build up of unacceptable stresses that could result in performance problems. Also, vertical spacing limitations in the AASHTO LRFD Specifications for MSE walls apply to walls designed using the K-Stiffness method.

13. Determine the length of the reinforcement required in the resisting zone by comparing the factored T_{max} value to the factored pullout resistance available as calculated per the AASHTO LRFD Specifications. If the length of the reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

15.5.3.1.5 Strength Limit State Design for Internal Stability Using the K-Stiffness Method – Steel Reinforced Walls

For steel reinforced soil walls, four strength limit states (soil failure, reinforcement rupture, connection rupture, and pullout) shall be evaluated for internal reinforcement strength and stiffness design. The design steps and related considerations are as follows:

1. Select a trial reinforcement spacing and steel area that is based on end-of-construction (EOC) conditions (i.e., no corrosion). Once the trial spacing and steel area have been selected, the reinforcement layer stiffness on a per ft of wall width basis, J_{EOC}, and wall global stiffness, S_{global}, can be calculated (**Equation 15-5**). Note that at this point in the design, it does not matter how one obtains the reinforcement spacing and area. They are simply starting points for the calculation. Also select a design soil friction angle to calculate K (see **Section 15.5.3.1.1**). Note that for steel reinforced wall systems, the reinforcement loads are not as strongly correlated to the peak plane strain soil friction angle as are the reinforcement loads in geosynthetic walls (**Allen and Bathurst, 2003**). This is likely due to the fact that the steel reinforcement is so much stiffer than the soil. The K-Stiffness Method was calibrated to a mean value of K_0 of 0.3 (this results from a plane strain soil friction angle of 44^o, or from triaxial or direct shear testing a soil friction angle of approximately 40^o). Therefore, soil friction angles higher than 44^o shall not be used. Lower design soil friction angles should be used for weaker granular backfill materials.
2. Begin by checking the strength limit state for backfill soil failure. The goal is to select a reinforcement density (spacing, steel area) that is great enough to keep the steel reinforcement load below yield ($A_sF_yR_c/b$, which is equal to A_sF_y/S_h). F_y is the yield stress for the steel, A_s is the area of steel before corrosion (EOC conditions), and S_h is the horizontal spacing of the reinforcement (use S_h = 1.0 for continuous reinforcement). Depending on the ductility of the steel, once the yield stress has been exceeded, the steel can deform significantly without much increase in load and can even exceed the strain necessary to cause the soil to reach a failure condition. For this reason, it is prudent to limit the steel stress to F_y for this limit state. Tensile tests on corroded steel indicate that the steel does not have the ability to yield to large strains upon exceeding F_y, as it does in an uncorroded state, but instead fails in a brittle manner (**Terre Armee, 1979**). Therefore, this limit state only needs to be evaluated for the steel without corrosion effects.
3. Using the trial steel area and global wall stiffness from Step 1, calculate the factored T_{max} for each reinforcement layer using **equations 15-3 and 15-4**.
4. Apply an appropriate resistance factor to A_sF_y/S_h to obtain the factored yield strength for the steel reinforcement. Then compare the factored load to the factored resistance, as shown in **Equation 15-18** below. If the factored load is greater than the factored yield strength, then increase A_s and recalculate the global wall stiffness and T_{max}. Make sure that the factored yield strength is greater than the factored load before going to the next limit state calculation. In general, this limit state will not control the design. If the yield strength available is well in excess of the factored load, it may be best to wait until the strength required for the other limit states has been determined before reducing the amount of reinforcement in the wall. Check to see that the factored reinforcement load T_{max} is greater than or equal to the factored yield resistance as follows:

$$T_{\max} \leq \frac{A_s F_y}{b} R_c \varphi_{sf} = \frac{A_s F_y}{S_h} \varphi_{\mathrm{sf}} \qquad (15\text{-}18)$$

   where φ_{sf} is the resistance factor for steel reinforcement resistance at yield, and S_h is the horizontal spacing of the reinforcement. For wire mesh, and possibly some welded wire mats with large longitudinal wire spacing, the stiffness of the reinforcement macro-structure could cause the overall stiffness of the reinforcement to be significantly less than the stiffness of the steel itself.

In-soil pullout test data may be used in that case to evaluate the soil failure limit state, and applied to the approach provided for soil failure for geosynthetic walls (see **Equation 15-14** in Step 5 for geosynthetic wall design).

5. Next, check the strength limit state for reinforcement rupture in the backfill. The focus of this limit state is to ensure that the long-term rupture strength of the reinforcement is greater than the load calculated from the K-Stiffness Method. Even though the focus of this calculation is at the end of the service life for the wall, the global stiffness for the wall should be based on the stiffness at the end of wall construction, as reinforcement loads do not decrease because of lost cross-sectional area resulting from reinforcement corrosion. T_{max} obtained from Step 5 should be an adequate starting point for this limit state calculation.
6. Calculate the strength of the steel reinforcement at the end of its service life, using the ultimate strength of the steel, F_u, and reducing the steel cross-sectional area, A_s, determined in Step 5, to A_c to account for potential corrosion losses. Then use the resistance factor φ_{rr}, as defined previously, to obtain the factored long-term reinforcement tensile strength such that T_{al} is greater than or equal to T_{max}, as shown below:

$$T_{al} = \frac{F_u A_c}{S_h}\varphi_{rr} \tag{15-19}$$

and,

$$T_{max} \leq \frac{F_u A_c}{b} R_c \varphi_{rr} = \frac{F_u A_c}{S_h}\varphi_{rr} \tag{15-20}$$

where, F_u is the ultimate tensile strength of the steel, and A_c is the steel cross-sectional area per FT of wall length reduced to account for corrosion loss. The resistance factor is dependent on the variability in F_u, A_s, and the amount of effective steel cross-sectional area lost as a result of corrosion. As mentioned previously, minimum specification values are typically used for design with regard to F_u and A_s. Furthermore, the corrosion rates provided in the AASHTO LRFD Specifications are also maximum rates based on the available data (**Terre Armee, 1991**). Recent post-mortem evaluations of galvanized steel in reinforced soil walls also show that AASHTO design specification loss rates are quite conservative (**Anderson and Sankey, 2001**). Furthermore, these corrosion loss rates have been correlated to tensile strength loss, so that strength loss due to uneven corrosion and pitting is fully taken into account. Therefore, the resistance factor provided in **Table 15-3**, which is based on the variability of the un-aged steel, is reasonable to use in this case, assuming that non-aggressive backfill conditions exist.

If T_{al} is not equal to or greater than T_{max}, increase the steel area, recalculate the global wall stiffness on the basis of the new value of A_s, reduce A_s for corrosion to obtain A_c, and recalculate T_{max} until T_{al} based on **Equation 15-20** is adequate to resist T_{max}.

7. If the steel reinforcement is connected directly to the wall facing (this does not include facings that are formed by simply extending the reinforcement mat), the reinforcement strength needed to provide the required long-term connection strength must be determined. This connection capacity, reduced by the appropriate resistance factor, must be greater than or equal to the factored reinforcement load at the connection. If not, increase the amount of reinforcing steel in the wall, recalculate the global stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.
8. Determine the length of reinforcement required in the resisting zone by comparing the factored T_{max} value to the factored pullout resistance available as calculated per Section 11 of the AASHTO LRFD specifications. If the length of reinforcement required is greater than desired (typically, the top of the wall is most critical), decrease the spacing of the reinforcement, recalculate the global wall stiffness, and re-evaluate all previous steps to ensure that the other strength limit states are met.

15.5.3.1.6 Seismic Design for Internal Stability Using the K-Stiffness Method

Seismic design of MSE walls when the K-Stiffness Method is used for internal stability design shall be conducted in accordance with Section 11.10.7.2 and 11.10.7.3 of the AASHTO LRFD Specifications, except that the static portion of the reinforcement load and associated load factors shall be for the K-Stiffness Method, and the resistance factors for combined seismic and static loading provided in **Table 15-3** shall be used.

15.5.3.1.7 Design Sequence Considerations for the K-Stiffness Method

A specific sequence of design steps has been proposed herein to complete the internal stability design of reinforced soil walls. Because global wall stiffness is affected by changes to the reinforcement design to meet various limit states, iterative calculations may be necessary. Depending on the specifics of the wall and reinforcement type, certain limit states may tend to control the amount of reinforcement required. It may therefore be desirable to modify the suggested design sequence to first calculate the amount of reinforcement needed for the limit state that is more likely to control the amount of reinforcement. Then perform the calculations for the other limit states to ensure that the amount of reinforcement is adequate for all limit states. Doing this will hopefully reduce the number of calculation iterations.

For example, for geosynthetic reinforced wrap-faced walls, with or without a concrete facia placed after wall construction, the reinforcement needed to prevent soil failure will typically control the global reinforcement stiffness needed, while pullout capacity is generally not a factor, and connection strength is not applicable. For modular concrete block-faced or precast panel-faced geosynthetic walls, the connection strength needed is likely to control the global reinforcement stiffness. However, it is also possible that reinforcement rupture or soil failure could control instead, depending on the magnitude of the stiffness of a given reinforcement product relative to the long-term tensile strength needed. The key here is that the combination of the required stiffness and tensile strength be realistic for the products available. Generally, pullout will not control the design unless reinforcement coverage ratios are low. If reinforcement coverage ratios are low, it may be desirable to evaluate pullout early in the design process. For steel strip, bar mat, wire ladder, and polymer strap reinforced systems, pullout often controls the reinforcement needed because of the low reinforcement coverage ratios used, especially near the top of the wall. However, connection strength can also be the controlling factor. For welded wire wall systems, the tensile strength of the reinforcement usually controls the global wall reinforcement stiffness needed, though if the reinforcement must be connected to the facing (i.e., the facing and the reinforcement are not continuous), connection strength may control instead. Usually, coverage ratios are large enough for welded wire systems (with the exception of ladder strip reinforcement) that pullout is not a controlling factor in the determination of the amount of reinforcement needed. For all steel reinforced systems, with the possible exception of steel mesh reinforcement, the soil failure limit state does not control the reinforcement design because of the very low strain that typically occurs in steel reinforced systems.

If the wall is located over a soft foundation soil or on a relatively steep slope, compound stability of the wall and slope combination may need to be evaluated as a service limit state in accordance with the AASHTO LRFD Specifications. It is recommended that this stability evaluation only be used to evaluate surfaces that intersect within the bottom 20 to 30% of the reinforcement layers. As discussed by **Allen and Bathurst (2003)**, available limit equilibrium approaches such as the ones typically used to evaluate slope stability do not work well for internal stability of reinforced soil structures, resulting in excessively conservative designs.

15.5.4 Prefabricated Modular Walls

Modular block walls without soil reinforcement, gabion, bin, and crib walls shall be considered prefabricated modular walls.

In general, modular block walls without soil reinforcement shall have heights no greater than 2.5 times the depth of the block into the soil perpendicular to the wall face, and shall be stable for all modes of internal and external stability failure mechanisms. In no case, shall their height be greater than 15 ft. Gabion walls shall be 15 feet or less in total height. Gabion baskets shall be arranged such that vertical seams are not aligned, i.e. baskets shall be overlapped.

15.5.5 Rock Walls

Rock walls shall be designed in accordance with the Standard Specifications, and the wall-slope combination shall be stable regarding overall stability as determined per **WSDOT GDM Chapter 7**.

Rock walls shall not be used unless the retained material would be at least minimally stable without the rock wall (a minimum slope stability factor of safety of 1.25). Rock walls are considered to act principally as erosion protection and they are not considered to provide strength to the slope unless designed as a buttress using limit equilibrium slope stability methods. Rock walls shall have a batter of 6V:1H or flatter. The rocks shall increase in size from the top of the wall to the bottom at a uniform rate. The minimum rock sizes shall be:

Depth from Top of Wall (ft)	Minimum Rock Size	Typical Rock Weight (lbs)	Average Dimension (in)
0	Two Man	200-700	18-28
6	Three Man	700-2000	28-36
9	Four Man	2000-4000	36-48
12	Five Man	4000-6000	48-54

Table 15-4 Minimum rock sizes for rock walls.

Rock walls shall be 12 feet or less in total height. Rock walls used to retain fill shall be 6 feet or less in total height if the rocks are placed concurrent with backfilling. Rock walls up to 12 feet in height may be constructed in fill if the fill is overbuilt and then cut back to construct the wall. Fills constructed for this purpose shall be compacted to 95% maximum density, per WSDOT Standard Specification Section 2-03.3(14)D.

15.5.6 Reinforced Slopes

Reinforced slopes do not have a height limit. However, reinforced slopes with a face slope greater than 1.2H:1V shall have a wrapped face or a welded wire face. Reinforcing shall have a minimum length of 6 feet. Turf reinforcement of the slope face shall only be used at sites where the average annual precipitation is 20 inches or more. Sites with less precipitation shall have wrapped faces regardless of the face angle. The primary reinforcing layers for reinforced slopes shall be vertically spaced at 3 feet or less. Primary reinforcement shall be steel, geogrid, or geotextile. The primary reinforcement shall be designed in accordance with **Elias, et al. (2001)**, using allowable stress design procedures, since LRFD procedures are not available. Secondary reinforcement centered between the primary reinforcement at a maximum vertical spacing of 1 ft shall be used, but it shall not be considered to contribute to the internal stability. Secondary reinforcement aids in compaction near the face. Gravel borrow shall be used for reinforced slope construction as modified by the General Special Provisions in Division 2. The design and construction shall be in accordance with the General Special Provisions.

The durability and corrosion requirements specified MSE walls in the AASHTO LRFD Bridge Design Specifications shall be used for reinforced slopes.

15.5.7 Soil Nail Walls

Soil Nail walls are not specifically addressed by the ASHTO LRFD Bridge Design Specifications. Soil nail walls shall be designed by the geotechnical designer using GoldNail version 3.11 or SNail version 2.11 or later computer programs and the following manuals:

- Lazarte, C. A., Elias, V., Espinoza, R. D., Sabatini, P. J., 2003. Geotechnical Engineering Circular No. 7, Soil Nail Walls, U.S. Department of Transportation, Federal Highway Administration, FHWA-IF-03-017, 305 pp.
- Byrne, R. J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G., 1996, Demonstration Project 103, Manual for Design & Construction Monitoring of Soil Nail Walls, Federal Highway Administration, FHWA-SA-96-069, 468 pp.
- Porterfield, J. A., Cotton, D. A., Byrne, R. J., 1994, Soil Nail Walls-Demonstration Project 103, Soil Nailing Field Inspectors Manual, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-93-068, 86 pp.

The LRFD procedures described in the Manual for Design & Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069 shall not be used.

When using SNail, the geotechnical designer should use the allowable option and shall pre-factor the yield strength of the nails, punching shear of the shotcrete, and the nail adhesion. Unfactored cohesion and friction angle shall be used and the analysis run to provide the minimum safety factors discussed above for overall stability.

When using GoldNail, the geotechnical designer should utilize the design mode and the safety factor mode of the program with the partial safety factors identified in the Manual for Design and Construction Monitoring of Soil Nail Walls, FHWA-SA-96-069.

The geotechnical designer shall design the wall at critical wall sections. Each critical wall section shall be evaluated during construction of each nail lift. To accomplish this, the wall shall be analyzed for the case where excavation has occurred for that lift, but the nails have not been installed. The minimum construction safety factor shall be 1.2 for noncritical walls and 1.35 for critical walls such as those underpinning abutments.

Permanent soil nails shall be installed in predrilled holes. Soil nails that are installed concurrently with drilling shall not be used for permanent applications, but may be used in temporary walls.

Soil nails shall be number 6 bar or larger and a minimum of 12 feet in length or 60 percent of the total wall height, whichever is greater. For nail testing, a minimum bond length and a minimum unbonded length of 5 feet is required. Nail testing shall be in accordance with the WSDOT Standard Specifications and General Special Provisions.

The nail spacing shouldl be no less than 3 feet vertical and 3 feet horizontal. In very dense glacially over consolidated soils, horizontal nail spacing should be no greater than 8 feet and vertical nail spacing should be no greater than 6 feet. In all other soils, horizontal and vertical nail spacing should be 6 feet or less.

Nails may be arranged in a square row and column pattern or an offset diamond pattern. Horizontal nail rows are preferred, but sloping rows may be used to optimize the nail pattern. As much as possible, rows should be linear so that each individual nail elevation can be easily interpolated from the station and elevation of the beginning and ending nails in that row. Nails that cannot be placed in a row must have station and elevation individually identified on the plans. Nails in the top row of the wall shall have at least 1 foot of soil cover over the top of the drill hole during nail installation. Horizontal nails shall not be used. Nails should be inclined at least 10 degrees downward from horizontal. Inclination should not exceed 30 degrees.

Walls underpinning structures such as bridges and retaining walls shall have double corrosion protected (encapsulated) nails within the zone of influence of the structure being retained or supported. All other nails shall be epoxy coated unless the wall is temporary.

15.6 Temporary Shoring

15.6.1 Overview

Temporary shoring and cut slopes are frequently used during construction of transportation facilities. The primary difference between temporary shoring and their permanent counterparts is their design life. Typically, the design life of temporary shoring is the length of time that the shoring or cut slope are required to construct the adjacent, permanent facility. Because of the short design life, temporary shoring is typically not designed for seismic loading, and corrosion protection is generally not necessary. Additionally, more options for temporary shoring are available due to limited requirements for aesthetics. Temporary shoring is most often designed by the contractor. The contractor is responsible for internal and external stability, as well as global slope stability, soil bearing capacity, and settlement of temporary shoring walls. Exceptions to this include shoring in unusual soil deposits or in unusual loading situations in which the State has superior knowledge and for which there are few acceptable options or situations where the shoring is supporting a critical structure or facility. One other important exception is for temporary shoring adjacent to railroads. Shoring within railroad right-of-way typically requires railroad review. Due to the long review time associated with their review, often 9 months or more, WSDOT has been designing the shoring adjacent to railroads and obtaining the railroad's review and concurrence prior to advertisement of the contract. Designers involved in alternative contract projects may want to consider such an approach to avoid construction delays.

The following sections discuss the purpose of temporary shoring, requirements for temporary cut slopes, and the types of available temporary shoring systems. Geotechnical data needed for design, factors influencing the choice of temporary shoring, and construction considerations are discussed below.

15.6.1.1 Purpose of Temporary Shoring

The primary purpose of temporary shoring is to safely support an excavation needed for a short period of time, typically the length of time needed to construct an adjacent permanent facility. Temporary shoring should be designed such that the risk to health and safety is kept to an acceptable level and that adjacent improvements are not damaged.

Examples of instances where temporary shoring may be necessary include:

- Support of an excavation until permanent structure is in-place;
- Control groundwater seepage; and
- Limit the extent of fill needed for preloads or temporary access roads/ramps.

Temporary shoring is used most often when excavation must occur adjacent to a structure or roadway and the structure or traffic flow cannot be disturbed. To determine if temporary shoring might be required for a project, a hypothetical 1:1 temporary excavation slope can be utilized to estimate likely limits of excavation for construction, unless the geotechnical designer recommends a different slope for estimating purposes. If the hypothetical 1:1 slope intersects roadway or adjacent structures, temporary shoring may be required for construction. Temporary shoring is generally the responsibility of the Contractor. The actual temporary slope used by the contractor for construction will likely be different than the hypothetical 1H:1V slope used during design to evaluate shoring needs. If the shoring is complex or critical to the operation of transportation facilities, WSDOT may design the shoring.

15.6.2 Temporary Cut Slopes

Temporary cuts slopes are used extensively in construction due to the ease of construction and low costs. Since the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. Because excavations are recognized as one of the most hazardous construction operations, temporary cut slopes must be designed to meet Federal and State regulations. Federal regulations regarding temporary cut slopes are presented in CFR Part 29, Sections 1926. The State of Washington regulations regarding temporary cut slopes are presented in Part N of the Washington Administrative Code (WAC) Section 296-155.

WAC 296-115 presents maximum allowable temporary cut slope inclinations based on soil or rock type, as shown in **Table 15-5**. WAC 296-115 also presents typical sections for compound slopes and slopes combined with trench boxes. The allowable slopes presented in the WAC are applicable to cuts 20 feet or less in height. Slopes inclinations steeper than those specified by the WAC or greater than 20 feet in height must be designed by a geotechnical engineer.

Soil or Rock Type	Maximum Allowable Temporary Cut Slopes (20 feet maximum height)
Stable Rock	Vertical
Type A Soil	¾H:1V
Type B Soil	1H:1V
Type C Soil	1½H:1V

Table 15-5 WAC 296-115 Allowable Temporary Cut Slopes

Type A Soil. Type A soils include cohesive soils with an unconfined compressive strength of 3000 psf or greater. Examples include clay and plastic silts with minor amounts of sand and gravel. Cemented soils such as caliche and glacial till (hard pan) are also considered Type A Soil. No soil is Type A if:

- It is fissured;
- It is subject to vibrations from heavy traffic, pile driving or similar effects;
- It has been previously disturbed;
- The soil is part of a sloped, layered system where the layers dip into the excavation at 4H:1V or greater; or
- The material is subject to other factors that would require it to be classified as a less stable material.

Type B Soil. Type B soils generally include cohesive soils with an unconfined compressive strength greater than 1000 psf but less than 3000 psf and granular cohesionless soils with a high internal angle of friction, such as angular gravel or glacially overridden sand and gravel soils. Some silty or clayey sand and gravel soils that exhibit an apparent cohesion may sometimes classify as Type B soils. Type B soils may also include Type A soils that have previously been disturbed, are fissured, or subject to vibrations. Soils with layers dipping into the excavation at inclinations steeper than 4H:1V can not be classified as Type B soil.

Type C Soil. Type C soils include most non-cemented granular soils (e.g. gravel, sand, and silty sand) and soils that do not otherwise meet Types A or B.

The allowable slopes described above apply to dewatered conditions. Flatter slopes may be necessary if seepage is present on the cut face or if localized sloughing occurs. Temporary cut slopes in excess of 20 feet in height or that are steeper than the allowable inclinations described above shall be designed by a registered civil engineer (geotechnical engineer) or licensed engineering geologist. **WSDOT GDM Chapter 10** provides requirements regarding the design and construction of cut slopes.

For open temporary cuts, the following requirements shall be met:

- No traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut.
- Exposed soil along the slope should be protected from surface erosion using waterproof tarps or plastic sheeting. If raveling of the slope faces becomes an issue, the exposed cuts should be faced with shotcrete.
- Construction activities should be scheduled so that the length of time the temporary cut is left open is reduced to the extent practical.
- Erosion control measures should be implemented as appropriate such that runoff from the site is reduced to the extent practical.
- Surface water should be diverted away from the excavation.
- The general condition of the slopes should be observed periodically by the Geotechnical Engineer or his representative to confirm adequate stability.

15.6.3 Types of Temporary Retaining Systems

15.6.3.1 Fill Applications

While most temporary retaining systems are used in cut applications, some temporary retaining systems are also used in fill applications. Typical examples include the use of MSE walls to support preload fills that might otherwise encroach into a wetland or other sensitive area, the use of modular block walls or wrapped face geosynthetic walls to support temporary access road embankments or ramps, and the use of temporary wrapped face geosynthetic walls to support fills during intermediate construction stages.

MSE walls, including wrapped face geosynthetic walls, are well suited for the support of preload fills because they can be constructed quickly, are relatively inexpensive, are suitable for retaining tall fill embankments, and can tolerate significant settlements. Modular block walls without soil reinforcement (e.g., ecology block walls) are also easy to construct and relatively inexpensive; however they should only be used to support relatively short fill embankments and are less tolerant to settlement than MSE walls. Therefore, block walls are better suited to areas with firm subgrade soils where the retained fill thickness behind the walls is less than 15 feet.

15.6.3.1.1 MSE Walls

MSE walls are described briefly in **WSDOT GDM Section 15.5.3** of this manual, and extensively in Publication No. FHWA-NHI-00-043 (**Elias, et al., 2001**). The governing design specifications for these walls are provided in the AASHTO LRFD Bridge Design Specifications (**2004**). Because the walls will only be in service a short time (typically a few weeks to a couple years), the reduction factors (e.g. creep, durability, installation damage, etc.) used to assess the allowable tensile strength of the reinforcing elements are typically much less than for permanent wall applications. The T_{al} values (i.e., long-term tensile strength) of geosynthetics, accounting for creep, durability, and installation damage in Appendix D of the WSDOT Qualified Products List (QPL) may be used for temporary wall design purposes. However, those values will be quite conservative, since the QPL values are intended for permanent reinforced structures. Alternatively, a default combined reduction factor for creep, durability, and installation damage in accordance with the AASHTO LRFD specifications may be used, ranging from a combined reduction factor RF of 4.0 for walls with a life of up to 3 years, to 3.0 for walls with a 1 year life, to 2.5 for walls with a 6 month life. If steel reinforcement is used for temporary MSE walls, the reinforcement is not required to be galvanized, and the loss of steel due to corrosion is estimated in consideration of the anticipated wall design life.

15.6.3.1.2 Prefabricated Modular Block Walls

Prefabricated modular block walls without soil reinforcement are discussed in **WSDOT GDM Section 15.5.4** of this manual and should be designed as gravity retaining structures. Concrete blocks used for gravity walls typically consist of 2½- by 2½- by 5-foot solid rectangular concrete blocks designed to interlock with each other. They are typically cast from excess concrete at concrete batch plants and are relatively inexpensive. Because of their rectangular shape they can be stacked a variety of ways. Because of the tightly fitted configuration of a concrete block wall, oversized blocks will tend to fit together poorly. Occasionally, blocks from a concrete batch plant are found to vary in dimension by several inches. The blocks shall meet the requirements in the WSDOT Standard Specifications. Implementation of this specification will reduce the difficulties associated with placing blocks in a tightly fitted manner. Large concrete blocks should not be placed along a curve. Curves should be accomplished by staggering the wall in one-half to one full block widths.

15.6.3.2 Common Cut Applications

A wide range of temporary shoring systems are available for cut applications. Each temporary shoring system has advantages and disadvantages, conditions where the system is suitable or not suitable, and specific design considerations. The following sections provide a brief overview of many common temporary shoring systems for cut applications. The "Handbook of Temporary Structures in Construction" (**Ratay, 1996**) is another useful resource for information on the design and construction of temporary shoring systems.

15.6.3.2.1 Trench Boxes

Trench boxes are routinely used to protect workers during installation of utilities and other construction operations requiring access to excavations deeper than 4 feet. Trench boxes consist of two shields connected by internal braces and have a fixed width and height. The typical construction sequence consists of excavation of a trench and then setting the trench box into the excavation prior to allowing workers to gain access to the protected area within the trench box. For utility construction, the trench box is commonly pulled along the excavation by the excavator as the utility construction advances. Some trench boxes are designed such that the trench boxes can be stacked for deeper excavations.

The primary advantage of trench boxes is that they provide protection to workers for a low cost and no site specific design is generally required. Another advantage is that trench boxes are readily available and are easy to use. One disadvantage of trench boxes is that no support is provided to the soils—where existing improvements are located adjacent to the excavation, damage may result if the soils cave-in towards the trench box. Therefore, trench boxes are not suitable for soils that are too weak or soft to temporarily support themselves. Another disadvantage of trench boxes is the internal braces extend across the excavation and can impede access to the excavation. Finally, trench boxes provide no cutoff for groundwater; thus, a temporary dewatering system may be necessary for excavations that extend below the water table for trench boxes to be effective.

Trench boxes are most suitable for trenches or other excavations where the depth is greater than the width of the excavation and soil is present on both sides of the trench boxes. Trench boxes are not appropriate for excavations that are deeper than the trench box. Generally, detailed analysis is not required for design of the system; however, the contractor should be aware of the trench box's maximum loading conditions for situations where surcharge loading may be present. Geotechnical information required to determine whether trench boxes are appropriate for an excavation include the soil type, density, and groundwater conditions. Also, where existing improvements are located near the excavation, the soil should exhibit adequate standup time to minimize the risk of damage as a result of caving soil conditions against the outside of the trench box. Much of the required information presented in **WSDOT GDM Section 15.3** is pertinent to the design of trench boxes.

15.6.3.2.2 Sheet Piling

Sheet piling is a common temporary shoring system in cut applications and is particularly beneficial as the sheet piles can act as a diaphragm wall to reduce groundwater seepage into the excavation. Sheet piling typically consists of interlocking steel sheets that are much longer than they are wide. Sheets can also be constructed out of vinyl, aluminum, concrete, or wood; however, steel sheet piling is used most often due to its ability to withstand driving stresses and its ability to be removed and reused for other walls. Sheet piling is typically installed by driving with a vibratory pile driving hammer. For sheet piling in cut applications, the piling is installed first, then the soil in front of the wall is excavated or dredged to the design elevation. There are two general types of sheet pile walls: cantilever and anchored/braced.

Sheet piling is most often used in waterfront construction; although, sheet piling can be used for many upland applications. One of the primary advantages of sheet piling is that it can provide a cutoff for groundwater flow and the piles can be installed without lowering the groundwater table. Another advantage of sheet piling is that it can be used for irregularly shaped excavations. The ability for the sheet piling to be removed makes sheet piling an attractive shoring alternative for temporary applications. The ability for sheet piling to be anchored by means of ground anchors or deadman anchors (or braced internally) allows sheet piling to be used where deeper excavations are planned or where large surcharge loading is present. One disadvantage of sheet piling is that it is installed by vibrating or driving; thus, in areas where vibration sensitive improvements or soils are present, sheet piling may not be appropriate. Another disadvantage is that where very dense soils are present or where cobbles, boulders or other obstructions are present, installation of the sheets is difficult.

The design of sheet piling requires a detailed geotechnical investigation to characterize the retained soils and the soil located below the base of excavation/dredge line. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, and groundwater conditions. In situations where lower permeability soils are present at depth, sheet piles are particularly effective at cutting off groundwater flow. Where sheet piling is to be used to cutoff groundwater flow, characterization of the soil hydraulic conductivity is necessary for design.

15.6.3.2.3 Soldier Piles

Soldier pile walls are frequently used as temporary shoring in cut applications. The ability for soldier piles to withstand large lateral earth pressures and the proven use adjacent to sensitive infrastructure make soldier piles an attractive shoring alternative. Soldier pile walls typically consist of steel beams installed in drilled shafts; although, drilled shafts filled with steel cages and concrete or precast reinforced concrete beams can be used. Following installation of the steel beam, the shaft is filled with structural concrete, lean concrete, or a combination of the two. The soldier piles are typically spaced 6 to 8 feet on center. As the soil is excavated from in front of the soldier piles, lagging is installed to retain the soils located between adjacent soldier piles. The lagging typically consists of timber; however, reinforced concrete beams, reinforced shotcrete, or steel plates can also be used as lagging. Ground anchors, internal bracing, rakers, or deadman anchors can be incorporated in soldier pile walls where the wall height is higher than about 12 feet, or where backslopes or surcharge loading are present.

Soldier piles are an effective temporary shoring alternative for a variety of soil conditions and for a wide range of wall heights. Soldier piles are particularly effective adjacent to existing improvements that are sensitive to settlement, vibration, or lateral movement. Construction of soldier pile walls is more difficult in soils prone to caving, running sands, or where cobbles, boulders or other obstructions are present; however, construction techniques are available to deal with nearly all soil conditions. The cost of soldier pile walls is higher than some temporary shoring alternatives. In most instances, the steel soldier pile is left in place following construction. Where ground anchors or deadman anchors are used, easements may be required if the anchors extend outside the right-of-way/property boundary. Where ground anchors are used and soft soils are present below the base of the excavation, the toe of the soldier pile should be designed to prevent excessive settlements.

Design of soldier pile walls requires a detailed geotechnical investigation to characterize the retained soils and the soil located below the base of excavation. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, surcharge loading, foreslope and backslope inclinations, and groundwater conditions. The required information presented in **WSDOT GDM Sections 15.3 and 15.5.3** is pertinent to the design of temporary soldier pile walls.

15.6.3.2.4 Modular Block Walls

In general, modular blocks (see **WSDOT GDM Section 15.6.3.1.2**) for cut applications require the soil deposit to have adequate standup time such that the excavation can be made and the blocks placed without excessive caving. Otherwise large temporary backcuts and subsequent backfill placement may be required. A key advantage to modular block walls is that the blocks can be removed and reused after the temporary structure is no longer needed. One disadvantage to using modular blocks in cut applications is that the blocks are placed in front of an excavation and the soils are initially not in full contact with the blocks unless the areas is backfilled. Some movement of the soil mass is required prior to load being applied to the blocks—this movement can be potentially damaging to upslope improvements.

Modular blocks are gravity retaining structures and as such, need to be designed to account for overturning, sliding, bearing capacity, settlement, and global stability. Adequate geotechnical information for the retained soils and the foundation soils is required to design a block wall. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, and groundwater conditions. Much of the required information presented in **WSDOT GDM Section 15.3** is pertinent to the design of modular block walls.

15.6.3.2.5 Braced Cuts

Braced cuts are used in applications where a temporary excavation is required that provides support to the retained soils in order to reduce excessive settlement or lateral movement of the retained soils. Braced cuts are generally used for trenches or other excavations where soil is present on both sides of the excavation and construction activities are not affected by the presence of struts extending across the excavation. A variety of techniques are available for constructing braced cuts; however, most include a vertical element, such as a sheet pile, metal plate, or a soldier pile, that is braced across the excavation by means of struts. Many of the considerations discussed below for soldier pile walls and sheet piling apply to braced cuts.

15.6.3.2.6 Soil Nail Walls

The soil nail wall system consists of drilling and grouting rows of steel bars or “nails” behind the excavation face as it is excavated and then covering the face with reinforced shotcrete. The placement of soil nails reinforces the soils located behind the excavation face and increases the soil’s ability to resist a mass of soil from sliding into the excavation. Soil nail walls are typically used in dense to very dense granular soils or stiff to hard, low plasticity, fine-grained soils. Soil nail walls are less cost effective in loose to medium dense sands or soft to medium stiff/high plasticity fine-grained soils.

The soils typically are required to have an adequate standup time (to allow placement of the steel wire mesh and/or reinforcing bars to be installed and the shotcrete to be placed). Soils that have short standup times are problematic for soil nailing. Many techniques are available for mitigating short standup time, such as installation of vertical elements (vertical soil nails or light steel beams set in vertical drilled shafts placed several feet on center along the perimeter of the excavation), drilling soil nails through soil berms, use of slot cuts, and flash-coating with shotcrete. Easements may be required if the soil nails extend outside the right-of-way/property boundary.

Design of soil nail walls requires a detailed geotechnical investigation to characterize the reinforced soils and the soil located below the base of excavation. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, surcharge loading, foreslope and backslope inclinations, and groundwater conditions. The required information presented in **WSDOT GDM Sections 15.3 and 15.5.4** is pertinent to the design of temporary soil nail walls.

15.6.3.3 Uncommon Shoring Systems for Cut Applications

The following shoring systems require special, very detailed, expert implementation, and will only be allowed either as a special design by the State, or with special approval by the State Geotechnical Engineer and State Bridge Engineer.

15.6.3.3.1 Diaphragm/Slurry Walls

Diaphragm/slurry walls are constructed by excavating a deep trench around the proposed excavation. The trench is filled with a weighted slurry that keeps the excavation open. The width of the trench is at least as wide as the concrete wall to be constructed. The slurry trench is completed by installing steel reinforcement cages and backfilling the trench with tremied structural concrete that displaces the slurry. The net result is a continuous wall that significantly reduces horizontal ground water flow. Once the concrete cures, the soil is excavated from in front of the slurry wall. Internal bracing and/or ground anchors can be incorporated into slurry walls. Diaphragm/slurry walls can be incorporated into a structure as permanent walls.

Diaphragm/slurry walls are most often used where groundwater is present above the base of the excavation. Slurry walls are also effective where contaminated groundwater is to be contained. Slurry walls can be constructed in dense soils where the use of sheet piling is difficult. Other advantages of slurry walls include the ability to withstand significant vertical and lateral loads, low construction vibrations, and the ability to construct slurry walls in low-headroom conditions. Slurry walls are particularly effective in soils where high groundwater and loose soils are present, and dewatering could lead to settlement related damage of adjacent improvements, assuming that the soils are not so loose or soft that the slurry is inadequate to prevent squeezing of the very soft soil.

In addition to detailed geotechnical design information, diaphragm/slurry walls require jobsite planning, preparation and control of the slurry, and contractors experienced in construction of slurry walls. For watertight applications, special design and construction considerations are required at the joints between each panel of the slurry wall. Considerations presented in **WSDOT GDM Section 15.3** are pertinent to the design of diaphragm/slurry walls.

15.6.3.3.2 Secant Pile Walls

Secant pile walls are another type of diaphragm wall that consist of interconnected drilled shafts. First, every other drilled shaft is drilled and backfilled with low strength concrete without steel reinforcement. Next, structural drilled shafts are installed between the low strength shafts in a manner that the structural shafts overlap the low strength shafts. The structural shafts are typically backfilled with structural concrete and steel reinforcement. The net result is a continuous wall that significantly reduces horizontal ground water flow while retaining soils behind the wall.

Secant pile walls are typically more expensive than many types of cut application temporary shoring alternatives; thus, the use of secant pile walls is limited to situations where secant pile walls are better suited to the site conditions than other shoring alternatives. Conditions where secant pile walls may be more favorable include high groundwater, the need to prevent migration of contaminated groundwater, sites where dewatering may induce settlements below adjacent improvements, sites with soils containing obstructions, and sites where vibrations need to be minimized.

The design of a secant pile wall requires a detailed geotechnical investigation to characterize the retained soils and the soil located below the base of excavation. The geotechnical information required for design includes soil stratigraphy, unit weight, shear strength, and groundwater conditions. Considerations presented in **WSDOT GDM Section 15.3** are pertinent to the design of secant pile walls.

15.6.3.3.3 Cellular Cofferdams

Sheet pile cellular cofferdams can be used for applications where internal bracing is not desirable due to interference with construction activities within the excavation. Cellular cofferdams are typically used where a dewatered work area or excavation is necessary in open water or where large dewatered heads are required. Cellular cofferdams consist of interlocking steel sheet piles constructed in a circle, or cell. The individual cells are constructed some distance apart along the length of the excavation or area to be dewatered. Each individual cell is joined to adjacent cells by arcs of sheet piles, thus providing a continuous structure. The cells are then filled with soil fill, typically granular fill that can be densified. The resulting structure is a gravity wall that can resist the hydrostatic and lateral earth pressures once the area within the cellular cofferdam is dewatered or excavated. As a gravity structure, cellular cofferdams need adequate bearing; therefore, sites where the cellular cofferdam can be founded on rock or dense soil are most suitable for these structures.

Cellular cofferdams are difficult to construct and require accurate placement of the interlocking sheet piles. Sites that require installation of sheet piles through difficult soils, such as through cobbles or boulders are problematic for cellular cofferdams and can result in driving the sheets out of interlock.

15.6.3.3.4 Frozen Soil Walls (Ground Freezing)

Frozen soil walls can be used for a variety of temporary shoring applications including construction of deep vertical shafts and tunneling. Frozen soil walls are typically used where conventional shoring alternatives are not feasible or have not been successful. Frozen soil walls can be constructed as gravity structures or as compressive rings. Ground freezing also provides an effective means of cutting of groundwater flows. Frozen soil has compressive strengths similar to concrete. Installation of a frozen soil wall can be completed with little vibration and can be completed around existing utilities or other infrastructure. Ground freezing is typically completed by installing rows of steel freeze pipes along the perimeter of the planned excavation. Refrigerated fluid is then circulated through the pipes at temperatures typically around -20°C to -30°C. Frozen soil forms around each freeze pipe until a continuous mass of frozen soil is present. Once the frozen soil reaches the design thickness, excavation can commence within the frozen soil.

Frozen soil walls can be completed in difficult soil and groundwater conditions where other shoring alternatives are not feasible. Frozen soil walls can provide an effective cutoff for groundwater and are well suited for containment of contaminated groundwater. Frozen soil walls are problematic in soils with rapid groundwater flows, such as coarse sands or gravels, due to the difficulty in freezing the soil. Flooding is also problematic to frozen soil walls where the flood waters come in contact with the frozen soil—a condition which can lead to failure of the shoring. Special care is required where penetrations are planned through frozen soil walls to prevent groundwater flows from flooding the excavation. Accurate installation of freeze pipes is required for deeper excavations to prevent windows of unfrozen soil. Furthermore, ground freezing can result in significant subsidence as the frozen ground thaws. If settlement sensitive structures are below or adjacent to ground that is to be frozen, alternative shoring means should be selected.

Design of frozen soil walls requires detailed geotechnical information including the soil stratigraphy, groundwater flow velocity, soil gradation and hydraulic conductivity, and soil unit weight and shear strength parameters for both the frozen soil and unfrozen soil.

15.6.3.3.5 Deep Soil Mixing

Deep soil mixing (DSM) is an in-situ soil improvement technique used to improve the strength characteristics of panels or columns of native soils. DSM utilizes mixing shafts suspended from a crane to mix cement into the native soils. The result is soil mixed panels or columns of improved soils. Two types of DSM walls can be constructed: gravity walls and diaphragm-type walls. Gravity type DSM walls consist of columns or panels of improved soils configured in a pattern capable of resisting movement of soil into the excavation. Diaphragm-type DSM walls are constructed by improving the soil along the perimeter of the excavation and inserting vertical reinforcement into the improved soil immediately after mixing cement into the soil. The result is a low permeability structural wall that can be anchored with tiebacks, similar to a soldier pile wall, where the improved soil acts as the lagging.

Advantages with deep soil mixing gravity walls include the use of the native soils as part of the shoring system and reduced or no reinforcement. However, a significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. Advantages with soil mixed diaphragm walls include the ability to control groundwater seepage, construction of the wall facing simultaneously with placement of steel soldier piles, and a thinner zone of improved soils compared to gravity DSM walls.

DSM walls can be installed top-down by wet methods where mechanical mixing systems combine soil with a cementitious slurry or through bottom up dry soil mixing where mechanical mixing systems mix pre-sheared soil with pneumatically injected cement or lime. DSM is generally appropriate for any soil that is free of boulders or other obstructions; although, it may not be appropriate for highly organic soils. DSM can be completed in very soft to stiff cohesive soils and very loose to medium dense granular soils.

15.6.3.3.6 Permeation Grouting

Permeation grouting involves the pressurized injection of a fluid grout to improve the strength of the in-situ soils and to reduce the soil's permeability. A variety of grouts are available—micro-fine cement grout and sodium silicate grout are two of the more frequently used types in permeation grouting. To be effective, the grout must be able to penetrate the soil; therefore, permeation grouting is not applicable in cohesive soils or granular soils with more than about 20 percent fines. Disadvantages of permeation grouting is the expense of the process and the high risk of difficulties. Permeation grouting, like ground freezing or jet grouting, can be used to create gravity retaining walls consisting of improved soils or can be used to create compression rings for access shafts or other circular excavations.

In addition to characterizing the soils gradation and stratigraphy, it is important to characterize the permeability of the soils to evaluate the suitability of permeation grouting.

15.6.3.3.7 Jet Grouting

Jet grouting is a ground improvement technique that can be used to construct temporary shoring walls and groundwater cutoff walls. Jet grouting can also be used to form a seal or strut at the base of an excavation. Jet grouting is an erosion based technology where high velocity fluids are injected into the soil formation to break down the soil structure and to mix the soil with a cementitious slurry to form columns of improved soil. Jet grouting can be used to construct diaphragm walls to cutoff groundwater flow and can be configured to construct gravity type shoring systems or compressive rings for circular shafts. Jet grouting is applicable to most soil conditions; however, high plasticity clays or stiff to hard cohesive soils are problematic for jet grouting.

Advantages with jet grouting include the ability to use of the native soils as part of the shoring system. A significant volume of the native soils needs to be improved over a wide area to enable the improved soil to act as a gravity structure. The width of the improved soil column is difficult to control, thus the final face of a temporary shoring wall may be irregular or protrude into the excavation.

15.6.4 Geotechnical Data Needed for Design

The geotechnical data needed for design of temporary shoring is essentially the same as needed for the design of permanent retaining structures. **WSDOT GDM Section 15.3.2** presents a bullet list of issues that should be addressed when developing the subsurface exploration and laboratory testing programs as well as considerations for shoring selection and design, such as risk, constructability, site constraints, and performance requirements.

WSDOT GDM Sections 15.3.3 through 15.3.6 discuss fieldwork and laboratory testing needs for permanent retaining structures. Ideally, the explorations and laboratory testing completed for the design of the permanent infrastructure will be sufficient for design of temporary shoring systems by the Contractor. This is typically the case. The exceptions may be if the selected temporary shoring system is very sensitive to groundwater flow velocities (e.g. frozen ground shoring) or if dewatering is anticipated during construction as the Contractor is also typically responsible for design and implementation of temporary dewatering systems. In these instances, there may need to be more emphasis on groundwater conditions at a site; multiple piezometers for water level measurements and a large number of grain size distribution tests on soil samples should be obtained. Downhole pump tests should be conducted if significant dewatering is anticipated, so the contractor has sufficient data to develop a bid and to design the system.

15.6.5 Factors Influencing Choice of Temporary Shoring

A multitude of factors will influence the choice of temporary shoring systems for a particular application. The most common considerations are cost, subsurface constraints (i.e. difficult driving conditions, the need to cutoff groundwater seepage, etc.), site constraints (i.e. limited access, impacts to adjacent infrastructure, etc.), and local practice. The sections below, while not all-inclusive, provide a brief discussion of several of the factors that influence selection of temporary shoring systems.

15.6.5.1 Application

The first screening criteria for alternative temporary shoring options will be the purpose of the shoring—will it retain an excavation or support a fill. As discussed in **WSDOT GDM Section 15.7.3.1**, the most frequently used temporary retaining systems for fill applications are MSE walls and modular block walls. The most frequently used temporary excavation support systems in Washington State include trench boxes for trench support, sheet piling, soil nailing, soldier piles, and braced sheeting.

15.6.5.2 Cut/fill Height

Some retaining systems are more suitable for supporting deep excavations/fill thicknesses than others. Temporary modular block walls are typically suitable only for relatively short fill embankments (less than 15 feet), while MSE walls can be designed to retain fills several tens of feet thick.

In cut applications, the common cantilever retaining systems (sheetpiling and soldier piles) are typically most cost effective for retained soil heights of 12 to 15 feet or less. Temporary shoring walls in excess of 15 feet typically require bracing, either external (struts, rakers, etc.) or internal (ground anchors or dead-man anchors).

15.6.5.3 Soil Conditions

15.6.5.3.1 Dense Soils and Obstructions

Dense subsurface conditions, such as presented by glacial till or bedrock, result in difficult installations conditions for temporary shoring systems that are typically driven or vibrated into place (sheet piling). Cobbles, boulders and debris within the soils also often present difficult driving conditions. It is often easier to use drilling methods to install shoring in these conditions. However, oversize materials and dense conditions may also hinder conventional auger drilling resulting in the need for specialized drilling equipment. Methods such as slurry trenches and grouting may become viable in areas with very difficult driving and drilling conditions.

15.6.5.3.2 Caving Conditions

Caving conditions caused by a combination of relatively loose cohesionless soils and/or groundwater seepage may result in difficult drilling conditions and the need to use casing to keep the holes open. Cased drilling, while routinely used in Washington State, is more expensive than uncased drilling.

15.6.5.3.3 Permeability

Soil permeability is based primarily on the soil grain size distribution and density. It influences how readily groundwater flows through a soil. If soils are very permeable and the excavation will be below the water level, then some sort of groundwater control will be required as part of the shoring system; this could consist of traditional dewatering methods or the use of shoring systems that also function as a barrier to seepage, such as sheet piling and slurry trench methods to name a few.

15.6.5.3.4 Bottom Heave and Piping

Bottom heave and piping can occur in soft/loose soils when the hydrostatic pressure below the base of the excavation is significantly greater than the resistance provided by the floor soils. In this case, temporary shoring systems that can be used to create a seepage barrier below the excavation, thus increasing the flow path and reducing the hydrostatic pressure below the base, may be better suited than those that do not function as a barrier. For example, sheet piling can be installed as a seepage barrier well below the base of the excavation, while soldier pile systems cannot. This is especially true if an aquitard is situated below the base of the excavation where the sheet piles can be embedded into the aquitard to seal off the groundwater flow path.

15.6.5.3.5 High Locked in Lateral Stresses

Glacially consolidated soils, especially fine-grained soils, often have high locked in lateral stresses because of the overconsolidation process (i.e. K_o can be much greater than a typical normally consolidated soil deposit). The Seattle Clay is an example of this type of soil, and much has been written about the performance of cuts into this material made to construct Interstate 5 (**Peck, 1963; Sherif, 1966; Andrews, et al., 1966; and Strazer, et al., 1974**). When cuts are made into soils with high locked in lateral stresses, they tend to rebound upon the stress relief, which can open up joints and fractures. Hydrostatic pressure buildup in the joints and fractures can function as a hydraulic jack and move blocks of soil, and movement can quickly degrade the shear strength of the soil. Therefore, for excavations into virgin material suspected of having high locked in lateral stresses, temporary shoring methods that limit the initial elastic rebound are required. For example, anchored shoring systems that are loaded and locked-off before the excavation will likely perform better than passive systems that allow the soil move, such as soil nails.

15.6.5.3.6 Compressible Soils

Compressible soils are more likely to impact the selection of temporary walls used to retain fills. As discussed above and in **WSDOT GDM Section 15.7.3.1**, MSE walls are typically more settlement tolerant than other fill walls, such as modular block walls.

15.6.5.4 Groundwater

The groundwater level with respect to the proposed excavation depth will have a substantial influence on the temporary shoring system selected. Excavations that extend below the groundwater table and that are underlain by relatively permeable soils will require either dewatering, shoring systems that also function as a barrier to groundwater seepage, or some combination thereof. If the anticipated dewatering volumes are high, issues associated with treating and discharge of the effluent can be problematic. Likewise, large dewatering efforts can cause settlement of nearby structures if they are situated over compressible soils, or they may impact nearby contamination plumes, should they exist. Considerations for barrier systems include the depth to an aquitard to seal off groundwater flow and estimated flow velocities. If groundwater velocity is high, some barrier systems such as frozen ground and permeation grouting will not be suitable.

15.6.5.5 Space Limitations

Space limitations include external constraints, such as right-of-way issues and adjacent structures, and internal constraints such as the amount of working space required. If excavations are required near existing right-of-ways, then temporary construction easements may be required to install the shoring system. Permanent easements may be required if the shoring systems include support from ground anchors or dead-man anchors that may remain after construction is complete. To minimize the need for temporary and permanent easements, cantilever walls or walls with external bracing (e.g. struts or rakers) should be considered. However, if the work space in front of the excavation needs to be clear, then shoring systems with external support may not be appropriate.

Existing infrastructure, such as underground utilities that cannot be relocated, may have the same impact on the choice of temporary shoring system as nearby right-of-ways.

15.6.5.6 Adjacent Infrastructure

The location of infrastructure adjacent to the site and the sensitivity of the infrastructure to settlement and/or vibrations will influence the selection of temporary shoring. For example, it may be necessary to limit dewatering or incorporate recharge wells if the site soils are susceptible to consolidation if the water table is lowered. If the adjacent infrastructure is brittle or supported above potentially liquefiable soils, it may be necessary to limit vibrations, which may exclude the selection of temporary shoring systems that are driven or vibrated into place, such as sheet piling.

The shoring system itself could also be sensitive to adjacent soil improvement or foundation installation activities. For example, soil improvement activities such as the installation of stone columns in loose to medium dense sands immediately in front of a shoring structure could cause subsidence of the loose sands and movement, or even failure, of the shoring wall. In such cases, the shoring wall shall be designed assuming that the soil immediately in front of the wall could displace significantly, requiring that the wall embedment be deepened and ground anchors be added.

15.6.6 General Design Considerations

15.6.6.1 Design Approach/Resistance Factors

The Contractor is responsible for the design of temporary shoring and shall use the AASHTO LRFD Bridge Design Specifications and as augmented herein for geotechnical design of shoring systems. For those wall systems that do not yet have a developed LRFD methodology developed, for example, soil nail walls, the FHWA design manuals identified herein that utilize allowable stress methodology shall be used. The design methodology, input parameters, and assumptions used must be clearly stated on the required submittals (see **WSDOT GDM Section 15.7.7**).

Regardless of the methods used, the temporary shoring design must consider both internal and external stability. Internal stability includes assessing the components that comprise the shoring system, such as the reinforcing layers for MSE walls, the bars or tendons for ground anchors, and the structural steel members for sheet pile walls and soldier piles to name a few. External stability includes an assessment of overturning, sliding, bearing resistance, settlement and global stability.

For temporary structures, the load and resistance factors provided in the AASHTO LRFD specifications are applicable. For soil nail walls, use the safety factors provided in the FHWA manuals identified herein. The resistance factor for global stability should be 0.65 if the temporary shoring system is supporting another structure (factor of safety of 1.5 for walls designed by Allowable stress) and 0.75 if the shoring system is not supporting another structure (factor of safety of 1.3 for walls designed by Allowable stress).

Estimates of settlement and associated impacts must be assessed as part of the design.

The primary difference between permanent retaining structures and temporary retaining structures is the design life of the structures. The design life of temporary shoring typically ranges from a few weeks to a couple years. Because of the short duration, temporary structures generally need not consider seismic loading and can utilize reduced corrosion/degradation design requirements (see **WSDOT GDM Section 5.7.3.1**).

15.6.6.2 Design Loads

The unfactored active, passive, and at-rest earth pressures loads used to design temporary shoring shall be determined in accordance with the procedures outlined in Article 3.11.5 of the AASHTO LRFD Specifications. Surcharge loads on temporary shoring shall be estimated in accordance with the procedures presented in Article 3.11.6 of the AASHTO LRFD Specifications. It is important to note that temporary shoring systems often are subject to surcharge loads from stockpiles and construction equipment and these surcharges loads can be significantly larger than typical vehicle surcharge loads often used for design of permanent structures. The design of temporary shoring must consider the actual construction-related loads that could be imposed on the shoring system. As described previously, temporary structures are typically not designed for seismic loads. Similarly, geologic hazards, such as liquefaction, are not mitigated for temporary shoring systems.

15.6.7 Construction Considerations

The contractor design for temporary shoring systems will need to be submitted to WSDOT for review by the Geotechnical Division and/or the Bridge and Structures Office depending upon the type of shoring system. The design shall be stamped by a licensed Civil Engineer registered in Washington State. The geotechnical elements of the design and the structural elements of the design shall be completed by Civil Engineers experienced in the fields of geotechnical engineering and structural engineering, respectively. As a minimum, the shoring submittal should include the following geotechnical information:

- All plan sheets, notes and specifications that depict the design.
- A summary clearly describing performance objectives, subsurface soil and groundwater conditions, site constraints, sequencing considerations, and governing assumptions.
- Supporting geotechnical calculations including the soil and material properties selected for design, and the justification for the selection for those properties.
- A monitoring plan.
- An estimate of expected displacements or vibrations, threshold limits that would trigger remedial actions, and a list of potential remedial actions should action levels be triggered.

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Appendices

Preapproved Wall Appendices